HYDRAULIC POWER DEVELOPMENT

ON

EAST CANADA CREEK, NEW YORK

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ARMOUR INSTITUTE OF TECHNOLOGY

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PROPOSED HYDRAULIC POWER DEVELOPMENT.

ON

EAST CANADA CREEK.

AT

INGHAM MILLS, NEW YORK.

A THESIS

Presented By

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To The

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OF

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INTRODUCTION.

Natural water powers have long held an important place among the sources of energy available for industrial purposes. Within the past few years, the progress made in the methods for converting mechanical into electrical energy, and the increase in the distance to which the latter can be economically transmitted, have led to the utilization of many water powers. The advantages to any community of cheap and reliable power are so great, that a steady growth of this kind is to be expected. Apart from manufacturers of all kinds, the purely municipal purposes of lighting and electric traction will of themselves, absorb a considerable amount of power.

The investigation of any water power project should include a careful study of all available data relating to the topographical and meteorological factors that effect the water supply and that obtain on the drainage area of the stream on which the proposed hydraulic power development is projected. The present condition of these factors is readily obtainable by careful observation and surveys, but the most difficult and yet the most important information needed for the correct understanding of the project, is the variations from the present conditions that have occurred in the past and that are therefore liable to re-occur in the future.

On the correct intrepretation of the available data the success of the project, or at least the economy of the installation, depends, especially if, as is usually the case, it is desired to develop the plant to its economical maximum.

The state of New York is richly endowed with natural advantages favorable to a full utilization of its vast water power resources. On its many rivers, especially those springing from the prolific water producing region of the Adirondacks, there are many advantageous sites for creating new developments, and for increasing by artificial storage the capacity of existing power plants.

Ingham Mills is about five miles from Little Falls, New York and is reached by a branch of the New York Central Railroad.

The situation of Ingham Mills is exceptionally advantageous for a hydraulic power plant, being in the center of a fairly well developed manufacturing district. Within a radius of thirty miles are Gloversville, Johnstown, Fonda and Canajoharie. The present use of electric current for lighting, traction and manufacturing purposes within this radius is great and the constantly increasing demand insures a large market.

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The object of this thesis is the profitable development

of the available water power of East Canada creek at Ingham Mills for the purpose of partially supplying this demand, and it is with the preliminary investigation, design and construction of such project that we shall confine ourselves.

HYDRAULIC CONDITIONS AND DEVELOPMENT.

The requisites of a reservoir site are numerous, among which may be mentioned the following:-

1. There must be an available water supply sufficient to fill the basin.

2. There must be a basin to hold this supply.

3. There must be a good dam site.

4. There must be suitable materials from which to construct a dam.

5. The foundation must be able to satisfactorily sustain the dam.

6. There must be available lands upon which to put the water.

7. The entire project must be on a commercial basis.

A comprehensive and detailed study of existing topographical maps and hydrographic data on the East Canada creek in the Mohawk system indicated promising opportunities for power development and storage.

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East Canada creek is the second important tributary of

the Mohawk. This creek rises in the south western part of Hamilton County, and flows southerly between Herkimer and Fulton Counties, joining the Mohawk at East creek, about seven miles from Little Falls. Its drainage area above Ingham Mills comprising approximately two hundred and seventy three square miles, contains about thirty small lakes and ponds and numerous swamps and marshes in the region of the head waters. A considerable part of the basin is timber covercd. The underlying rock is granitic gneiss in the upper portion of the basin, with limestone in some places. Heavy accumulations of snow occur during the winter.

The principal tributary of East Canada creek is Big Sprite creek, which is the outlet of the East Canada Lakes. The distance from the East Canada Lake outlet to its junction with East Canada creek, is about nine miles. In the first four miles there is a fall of three hundred and ninety feet. The remaining five miles to its mouth, has a fall of two hundred and forty-five feet.

The second tributary of East Canada creek is Spruce creek, which enters it one mile above Dolgeville and drains an area of fifty square miles. The total length from its source in the Eaton Mill pond to its mouth, is about nine miles; the total fall in this distance being approximately five hundred and fifty feet. Just below the Eaton Mill

pond there is a fall of one hundred and eighty feet in two thousand feet. At Salisbury Center, Spruce creek falls eighty-five feet in about nine hundred feet. A number of water power privileges are developed at this point. There is a total of twelve small dams on Spruce creek giving an aggregate fall of about one hundred and eighty feet.

The mean annual rainfall on the East Canada water-shed above Ingham Mills for the past ten years, as disclosed by studies of the United States Weather Bureau reports, is 40 inches.

The conditions on which the proposed development is based were obtained from the various water supply papers of the United States Geological Surve y for the past ten years (1899-1909 inclusive), containing the maximum and minimum daily and monthly discharges of the stream at Dolgeville about three miles up stream from Ingham Mills. At this point the discharge of the stream over the dam of the Herkimer County Light and Power Company, is computed from a discharge curve based on United States Geological Survey experiments.

About a mile above Ingham Mills, Gillette creek having a drainage area of approximately seventeen square miles, flows into East Canada creek. The drainage area of the East Canada creek above Dolgeville is two hundred and fifty-six square miles; thus the total area drained by the stream above

Ingham Mills is two hundred and seventy-three sq uare miles.

The data as to the discharge at Ingham Mills, were determined by multiplying the various discharge measurements obtained at Dolgeville by the constant $\frac{273}{256} = 1.06$; 273 being the total drainage area in square miles above Ingham Mills and 256 being the area above Dolgeville. This factor took into account the additional discharge due to Gillette creek. From these data mean monthly discharge curves were plotted. A curve was also plotted showing the daily discharge of the stream for the year 1906, which was a year of relatively high flood.

The flow of the stream at Ingham Mills varies from a maximum of 3280 second feet to a minimum of 90 second feet. A dam will be constructed at Stewart Landing near the head of Big Sprite creek by means of which a large service reservoir of 500,000,000 gallons capacity will be created. The function of this reservoir is to augment considerably the minimum discharge of the stream at Ingham Mills.

By constructing a dam across the valley where the stream is about ninety feet wide, a head of one hundred and twenty-five feet can be utilized. It was concluded that using a flow of two hundred and fifty second feet for twenty-four hours, or six hundred second feet for a period of ten hours the maximum power of the stream could be developed throughout the entire year without the necessity of an emergency steam

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plant.

The available power was determined as follows:

Let Q = discharge in second feet.

H = available head.

W = weight of a cubic foot of water

E = efficiency of water wheels

then Horse Power $= \dfrac{Q \times H \times W \times E}{550}$

or

$$\text{H.P.} = \frac{600 \times 125 \times 62.5 \times .85}{550} = 7200$$

The proposed development consists of a gravity dam, one hundred and twenty-five feet in height. The main conduit to be a pipe line with an internal diameter of twelve feet. The total length is eleven hundred feet of riveted steel pipe provided with a surge tank fifty eight feet high and forty feet in diameter. The function of the surge tank is to relieve the pipe line of excessive pressures due to a sudden closing of the gates in the water wheels, following a quick drop in the demand for power and also to maintain pressures and speed regulation in the station when sudden demands are made for water. The top of the surge tank rises to twenty-five feet above high water level above dam. From the surge tank there will be three riveted steel pipes, each six feet in diameter and four hundred feet in length, leading directly to the power house. The total hydrostatic head from the flow line of the reservoir to centers of receivers, will be one hundred and twenty-five feet.

DESIGN OF DAM.

A topographical survey using the stadia method, was made of the valley in the immediate vicinity of Ingham Mills. The general location of the dam was determined by the topography of the valley, but it required considerable study to fix its exact site. A number of test pits were sunk to bed rock in the stream by means of cofferdam construction. Limestone of a fairly compact texture was encountered.

The location chosen combines the advantages of a fairly short dam with a comparatively high position of bed rock. The cross section is shown on Plate 3.

The dam will be built of solid concrete masonry and founded on bed rock. Its lemgth measured on the crest is five hundred and ten feet. It will be one hundred and twenty-three feet above the present river bed and its foundation will extend two feet lower, making a total height of one hundred and twenty-five feet above bed rock with a batter of 1 in 12.5. Along the west bank of the stream at the site of the dam, there is a pocket about ten feet deep and five feet wide, caused by the erosive action of the stream at this point. In order to eliminate an unnecessary amount of rock excavation, this pocket will be filled with concrete.

At each end of the dam concrete core-walls, having a batter of 1 in 20 and placed two feet in rock extend into the adjoining sides of the valley in such a manner as to form

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At each end of the dam concrete core-walls, having a batter of 1 in 20 and placed two feet in rock extend into the adjoining sides of the valley in such a manner as to form

a perfectly safe seal and thus prevent the water from escaping around the ends of the structure.

A spillway for carrying off flood water will be constructed on the east end of the dam. The length of this waste-way is two hundred and sixty feet and its crest is ten feet lower than that of the retaining section. It is twenty-four feet in height, having a vertical up-stream face and an ogee down-stream face. Concrete of a 1-2-4 mixture will be used in constructing both the dam and spillway.

An intake for supplying the conduit will be placed near the west end of the dam on the up-stream side. At this intake the water is controlled by a twelve foot circular sluice gate. This gate has a non-rising stem, which passes through an operating stand on top of the dam. The stand is geared so that it may be operated by hand. Screens inclined at an angle of approximately sixty degrees to the horizontal, are placed at the entrance of the intake to protect the intake pipe. Provision is made for repairing the sluice gate by means of emergency stop logs. Access can be had to the sluice gate chamber by an iron ladder projectiong from the concrete wall and extending from the top to the pit of the chamber. Drainage of the chamber is provided by a twelve inch drain pipe, extending under the intake pipe and through to the down-stream side of the dam.

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The dam is designed purely as a gravity structure, and the usual standards of strength and stability are followed. The water was assumed to extend from the top of the dam to bed rock, but was not supposed to penetrate beneath and exert an upward pressure.

The profile as shown on Plate 3 is divided into six sections by horizontal joints. The centers of gravity of courses 1 to 6 are found in the usual manner, the vertical section of each course being assumed to form a trapezoid.

The retaining section was designed as follows:-

Reservoir Empty: The line of pressure was determined in the following manner:- The center of gravity of sections 1 and 2 are joined by a line and the center of gravity of these two combined sections found, as shown on Plate 3 . A line is drawn vertically downward from the center of gravity of sections 1 and 2. Where this vertical line intersects the line bb is one point in the line of pressure. In a similar manner the points where this line intersects the other joints are found. When the reservoir is empty, the only forces acting on the dam, are the weights of the different courses 1 to 6, each being applied at the center of gravity of the respective course.

Reservoir Full: The line of resultant pressure was determined as follows:- The pressure of the water acts at a point equal to one third the height from the base of section 2. This distance

is laid off to scale on the line drawn vertically downward through the center of gravity of sections 1 and 2. From this point a perpendicular line equal in length to the water pressure , is drawn. A line equal in length to the weight of the masonry, is drawn perpendicular to the line of water pressure. Where the resultant of the line representing the water pressure and the weight of masonry respectively, intersects the joint bb is one point in the line of resultant pressure. Similarly, the points where this resultant intersects the other joints are found.

The resultant of all joints is kept within the middle third, so that there are no tensile stresses.

The factors of safety are the following: As to Overturning:- The moment of the forces, which resist over-turning when taken about the down-stream edge of the dam at any elevation are more than twice as great as the moment of over-turning at the same point.

As to Sliding:- A coefficient of friction of .85 in the concrete masonry was assumed. Conservative engineering practice shows that this factor is considered amply safe in case of a concrete dam in which there will be no joints properly speaking, but on the contrary, considerable cohesive strength.

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As to Sliding:- A coefficient of friction of .65 in the concrete masonry was assumed. Conservative engineering practice shows that this factor is considered safe, and in case of a concrete dam in which there will be no joints, properly speaking, but on the contrary, considerable cohesion exists.

Spillway Section: The curve on the down-stream side was determined by plotting the parabolic curve, using the equation of the parabola

$$X^2 = 2\,P\,Y. \qquad (1)$$

where 2 P by experiment was found to be equal to 1.98 H; the height of water flowing over the crest of the spillway being assumed as five feet, then $H = \frac{H}{8} + 5$

$$\therefore \quad H = 5.710.$$

substituting the above values in (1)

Therefore: $X^2 = (1.98 \times 5.71)\,Y.$

The line of resultant pressure of the spillway section, the profile of which is shown on Plate 4 was determined in the same manner as that followed in the design of the retaining section.

In order to increase the height of the spillway section during low water, so as to dispose of a higher head than would be otherwise possible at a moment when such increase of head is most opportune , flash boards were designed so as to give way when the water reaches the desired elevation. These flash boards are held in place by "Wayne" iron pins fitted in pipe sleeves in the concrete. When a fiber stress of 69500 pounds per square inch is reached, the pins fail.

CALCULATIONS.

RETAINING SECTION:-

Area of section 1: $\frac{14}{2}\left[12+13.1\right] = 175.7$ sq. ft.

Area of section 2: $\frac{11}{2}\left[13.1+18\right] = 171$ sq. ft.

Weight of sections 1 and 2: $\left[175.7+171\right]\ 150 = 520123$ lbs/cu.ft.

Water pressure at 25 foot depth: $\frac{wh^2}{2} = \frac{62.5\times(25)^2}{2} = 19530$ lbs/sq.ft.

Resultant water pressure: 56000 lbs. per square foot.

Area of section 3: $\frac{25}{2}\left[18+34\right] = 650$ sq. ft.

Weight of sections 1, 2 and 3: $\left[175.7+171+650\right]\ 150 = 149513$ lbs./cu.ft

Water pressure at 50 foot depth: $\frac{62.5\times\overline{50}^2}{2} = 83125$ lbs./sq. ft.

Resultant pressure: 170400 lbs. per square foot

Area of section 4: $\frac{25}{2}\left[34+52\right] = 1075$ sq. ft.

Weight of sections 1, 2, 3 and 4: $\left[996.75+1075\right]\ 150 = 31076$ lbs./cu.ft.

Water pressure at 75 foot depth: $\frac{62.5\times\overline{75}^2}{2} = 175780$ lbs./sq. ft.

Resultant pressure: 357600 lbs. per square foot

Area of section 5: $\frac{25}{2}\left[52+70\right] = 1525$ sq. ft.

Weight of sections 1, 2, 3, 4 and 5: $\left[2071.75+1525\right]\ 150 = 539513$ lbs

Water pressure 100 foot depth: $\frac{62.5\times\overline{100}^2}{2} = 312500$ lbs./sq. ft.

Resultant pressure: 623500 lbs. per square foot.

Area of section 6: $\frac{25}{2}\left[70+88\right] = 1975$ sq. ft.

Weight of sections 1, 2, 3, 4, 5 and 6: $\left[3596.75+1975\right]\ 150 = 835763$

Water pressure 125 foot depth: $\frac{62.5\times\overline{125}^2}{2} = 488280$ lbs/sq. ft.

Total resultant pressure: 966700 lbs. per square foot.

RETAINING SECTION (concluded):-

Factor of safety against over-turning:

$$\frac{\text{Resisting Moment}}{\text{Overturning Moment}} = \frac{835763 \times 55.8}{488280 \times 41.7} = 2.2$$

Factor of safety against sliding:

Assuming coefficient of friction of concrete on rock as .85

$$\frac{835763 \times .85}{488280} = 1.4$$

$$p_1 = \frac{R}{l} = \frac{966700}{75.8} = 12753 \text{ lbs.}$$

$$p_2 = \frac{6Rb}{l^2} = \frac{6 \times 966700 \times 10.8}{(75.8)^2} = 10902 \text{ lb}$$

$$p_1 - p_2 = 12753 - 10902 = 1851^{\#} \text{ min.}$$

$$p_1 + p_2 = 12753 + 10902 = 23655^{\#} \text{ ma}$$

SPILLWAY SECTION:-

Area of section 1: $9 \times 1.43 = 12.87$ sq. ft.

Area of section 2: $\frac{2}{3}\left[9 \times 10.08\right] = 60.48$ sq. ft.

Area of section 3: $\frac{15}{2}\left[20.5 + 11.5\right] = 240$ sq. ft.

Total weight of sections 1, 2 and 3:

$$\left[12.87 + 60.48 + 240\right] 150 = 47000 \text{ lbs.}/\text{cu. ft.}$$

Total water pressure: $\frac{62.5 \times \overline{24}^2}{2} = 25600$ lbs./sq. ft.

Let h = height of section

d = depth of water on crest

Then point of application of water pressure is $\frac{h+3d}{h+2d} \times \frac{h}{3}$ from the bottom of section. $\frac{24+15}{24+10} \times 8 = 9.17'$

Total resultant pressure: 52200 lbs. per sq. ft.

Factor of safety against over-turning:

$$\frac{\text{Resisting Moment}}{\text{Overturning Moment}} = \frac{47000 \times 13.2}{25600 \times 9.17} = 2.6$$

SPILLWAY SECTION (concluded):-

Factor of safety against sliding:

Assuming coefficient of friction of concrete on rock as .85

$$\frac{47000 \times .85}{25600} = 1.5$$

$$p_1 = \frac{R}{l} = \frac{52200}{18} = 2900 \text{ lbs.}$$

$$p_2 = \frac{6Rb}{l^2} = \frac{6 \times 52200 \times 1.85}{(18)^2} = 1788 \text{ lbs.}$$

$$p_1 - p_2 = 2900 - 1788 = 1112 \text{ lbs. (minimum)}$$

$$p_1 + p_2 = 2900 + 1788 = 4688 \text{ lbs. (maximum)}$$

PIPE-LINE

The water is delivered to the power house through a riveted steel pipe, twelve feet in diameter and eleven hundred feet long, and through two six foot pipes which act as penstocks for the turbines in the power house. The connection between the single large pipe and the two smaller ones is made by means of a surge tank forty feet in diameter and fifty-eight feet high, which is placed on a high spot about four hundred feet from the power house.

The intake of this twelve foot pipe is at elevation 640, the crest of the dam at 675, giving a head of thirty-five feet, and was designed to carry twelve hundred cubic feet of water per second.

Using the formula $h = f\frac{l}{d} \times \frac{v^2}{2g}$, where

h = loss of head

f = coefficient

l = length

d = diameter

v = velocity

g = gravity

and assuming a 12 foot pipe

$$h = .01\ \frac{1050}{12} \times \frac{112.36}{64.3}$$

since $Q = AV$

$$V = \frac{Q}{A} = \frac{1200}{113} = 10.6$$

therefore h = 1.53

Slope, $S = \frac{h}{l} = \frac{1.53}{1050} = .00145$

Then from

$$V = c\sqrt{rS}$$

where c = constant

r = hydraulic mean radius: for full pipe $r = \frac{d}{4}$, since pipe must be designed for maximum.

$S = h/l$

Substituting values

$$v = 190\sqrt{3 \times .00145}$$

$$12.35 = \text{feet per second.}$$

This velocity is not too large because maximum amount of water will not be used and consequently the velocity will not reach this figure.

There fore use 12 foot pipe

Thickness of pipe:

Allowable tensile strength of pipe = 16000 lbs. per square inch

Head = 35 feet

Head (in feet) x .434 = lbs. per square inch

Then $W \times \frac{d}{2} = t \times 16000$;

$$35 \times .434 \times 6 \times 12 = t \times 16000 \times 12$$

$$\therefore t = .054 = \frac{1}{16}''$$

Allowing for water hammer, which occurs and also for stiffness of pipe use 3/8 inches.

Therefore Pipe Dimensions are as follows:-

Diameter:- 12 feet

Thickness:- 3/8 inches

Length of Plate:- 6 feet

RIVETS:-

Rivets should be $1\frac{1}{4}$ x thickness of thickest plate + $\frac{3}{16}$: then $\frac{5}{4} + \frac{3}{8} = \frac{15}{32} + \frac{6}{32} = \frac{21}{32}$; Therefore use $\frac{3}{4}''$ rivets.

The spacing should be not less than $1\frac{1}{2}$ x diameter + $\frac{1}{2}$
$\frac{3}{2} \times \frac{3}{4} = \frac{9}{8} \times \frac{4}{8} = \frac{13}{8} = 1\frac{5}{8}''$ use $2''$.

The girth seams are lap jointed, single riveted and the longitudinal seams lap jointed, double riveted. Considering six foot section, the pressure is found to be

1094 x 6 x 12 = 78720 lbs.

Allowing 12000 lbs. for double shear, then $\frac{78720}{12000} = 7^{+}$; therefore 8 rivets per foot will be used.

The diameters of adjacent 6 foot lengths vary by twice the thickness of the plate, forming inside and outside sections alternately, and angle irons $5'' \times 3''$ are placed at every other joint, thus strengthening the pipe between supports. The supports being concrete piers every 12 feet except through one section where they are placed every 10 feet. At this section the pipe is suspended 15 feet and the concrete piers are 20 feet high, reaching to the center line of the pipe. They are 2 feet thick and 16 feet wide.

The alignment of the pipe in a vertical plane consists of one long tangent about 1100 feet in length, running from the dam to the surge tank at the constant elevation of 640, and under a head of 35 feet. The drop in about 400 feet from the tank to the power house, is 90 feet from center line of pipe to center line of turbines, making a total head of 125 feet at the turbine.

Horizontally, there is a tangent 300 feet long running from the dam, then at an angle of about 62° another tangent 800 feet long to center line of tank. An expansion joint is provided fifteen feet from the end of first section; the end of the curve on first section being covered sufficiently deep to do away with expansion and contraction. The upper end of the first section has a fixed connection with the dam, and the lower end of the second section is rigidly connected to the tank.

At the expansion joint the shell of the adjacent pipe sections are increased from $\frac{3}{8}$ inches to $\frac{1}{2}$ inches; one pipe enters the other for not more than 30 inches, the rivets in the engaged section being counter-sunk. Between the inner pipe, and a sleeve 12 x $\frac{1}{2}$ inches, which project beyond the outer pipe, a packing such as hemp will be used, and secured by small plates riveted or bolted to the flange of the sleeve. Similar joints are provided for by elbows in the smaller pipes leading from the tank to the power house.

The alignment of the pipe in a vertical plane consists of one long tangent about 1100 feet in length, running from the dam to the surge tank at the constant elevation of 640, and under a head of 35 feet. The drop in about 400 feet from the tank to the power house, is 90 feet from center line of pipe to center line of turbines, making a total head of 125 feet at the turbine.

Horizontally, there is a tangent 300 feet long running from the dam, then at an angle of about 62° another tangent 800 feet long to center line of tank. An expansion joint is provided fifteen feet from the end of first section; the end of the curve on first section being covered sufficiently deep to do away with expansion and contraction. The upper end of the first section has a fixed connection with the dam, and the lower end of the second section is rigidly connected to the tank.

At the expansion joint the shell of the adjacent pipe sections are [illegible] one pipe enters the other for not more than 30 inches, the rivets in the engaged section being counter-sunk. Between the inner pipe, and a sleeve 12 x ½ inches, which project beyond the outer pipe, a packing such as hemp will be used, and secured by small plates riveted or bolted to the flange of the sleeve. Similar joints are provided for by elbows in the smaller pipes leading from the tank to the power house.

SURGE TANK:

The surge tank is used to take up water hammer occurring when the turbines are shut off, and saving the pipe, which might be bursted. The surge tank will be built 15 feet above crest of the dam, making the elevation of top of tank 690.

The tank was designed in three sections as follows:

Let w = weight of cubic foot of water = 62.5

h = height of section to be designed

d = diameter

A_s = area of steel

f_s = unit stress allowable = 16000

Therefore, for the first section (twenty-three feet)

$$62.5 \times 58 \times 20 = A_s \times 16000$$

$$A_s = 4.5 \text{ square inches}$$

$$A_s = .358 = \tfrac{5}{16} \text{ inches}$$

Allowing for water hammer, make plate ½ inch thick.

Second section (twenty feet)

$$62.5 \times 35 \times 20 = A_s \times 16000$$

$$A_s = 2.73 \text{ sq. ins} \qquad \therefore t = \frac{2.73}{12} = .23 = \tfrac{1}{4}''$$

Allow 3/16 inches for water hammer, plate will be 7/16 inches thick.

Third section (fifteen feet)

$$62.5 \times 15 \times 20 = A_s \times 16000$$

$$A_s = 1.17 \text{ sq. in.} \qquad \therefore t = \frac{1.17}{12} = .093''$$

For stiffness and safety, the thickness of this section will be made 3/8 of an inch.

Rivets for tank:

$\frac{5}{4} \times \frac{1}{2} = \frac{5}{8}$; $\frac{5}{8} + \frac{3}{16} = \frac{13}{16}$ $\therefore$ use $\frac{7}{8}$" rivets.

The spacing should be not less than

$\frac{3}{2} \times \frac{7}{8} = \frac{21}{16} + \frac{8}{16} = \frac{29}{16} =$ 2 inches. (approx.)

The stress at the bottom must be equal for every seam, then 62.5 x 55 x 20 = 68750 lbs. The allowable stress on each rivet is 7500 lbs. per square inch. Then $\frac{68750}{7500} = 9^{+}$; therefore 10 rivets are required. The girth seams are riveted two inches to make a water-tight connection.

POWER-HOUSE.

The power-house will be placed on the present site of an old grist mill, located on the north bank of the stream at a distance of about fifteen hundred and fifty feet from the site of the dam. The power-house will be built of concrete both substructure and superstructure. Its outside dimensions are 103 feet 4 inches x 49 feet 4 inches. The roof trusses are of steel and supported on concrete pilasters. The covering consists of corrugated red tile roofing. A traveling crane of twenty tons capacity, operated by hand power, traverses the building, the track girders being carried by the concrete pilasters. This building contains all the hydraulic units and electrical appurtenances, offices, and machine shop. The turbine units, transformers and machine shop are placed on the first floor. The switch-board gallery is placed along the south end of the building, and the top floor is reserved for bus bars and switch cells.

The available horse-power (7200) will be developed by three horizontal shaft S. Morgan Smith turbines each fifty-four inches in diameter, with a capacity of 2400 h.p. at 360 r.p.m. Each unit carries its own exciter unit mounted on same shaft. The turbines are each direct connected to three phase alternating currect generators, having an out-put of 1800 k.w. at 360 r.p.m.

The transformers are situated on the main floor along the east side of the power-house. They receive the current

POWER-HOUSE.

The power-house will be placed on the present site of an old grist mill, located on the north bank of the stream at a distance of about fifteen hundred and fifty feet from the site of the dam. The power-house will be built of concrete both substructure and superstructure. Its outside dimensions are 103 feet 4 inches x 49 feet 4 inches. The roof trusses are of steel and supported on concrete pilasters. The covering consists of corrugated red tile roofing. A traveling crane of twenty tons capacity, operated by hand power, traverses the building, the track girders being carried by the concrete pilasters. This building contains all the hydraulic units and electrical appurtenances, offices, and machine shop. The turbine units, transformers and machine shop are placed on the first floor. The switch-board gallery is placed along the south end of the building, and the top floor is reserved for bus bars and switch cells.

The available horse-power (7200) will be developed by three horizontal shaft S. Morgan Smith turbines each fifty-four inches in diameter, with a capacity of 2400 h.p. at 300 r.p.m. Each unit carries its own exciter unit mounted on same shaft. The turbines are each direct connected to three phase alternating current generators, having an output of 1800 K.W. at 300 r.p.m.

The transformers are situated on the main floor along the west side of the power-house. They receive the current

from the generators, and step up the voltage to 33000 at which pressure the currect will pass into the transmission lines.

For the present, two units of twenty-four hundred h.p. each are to be installed and provision in size of pen-stocks, power-house etc. left for a third unit of twenty-four hundred h.p. to be installed latter when increased market demand shall justify same.

Drainage Area of East Canada Creek Shaded

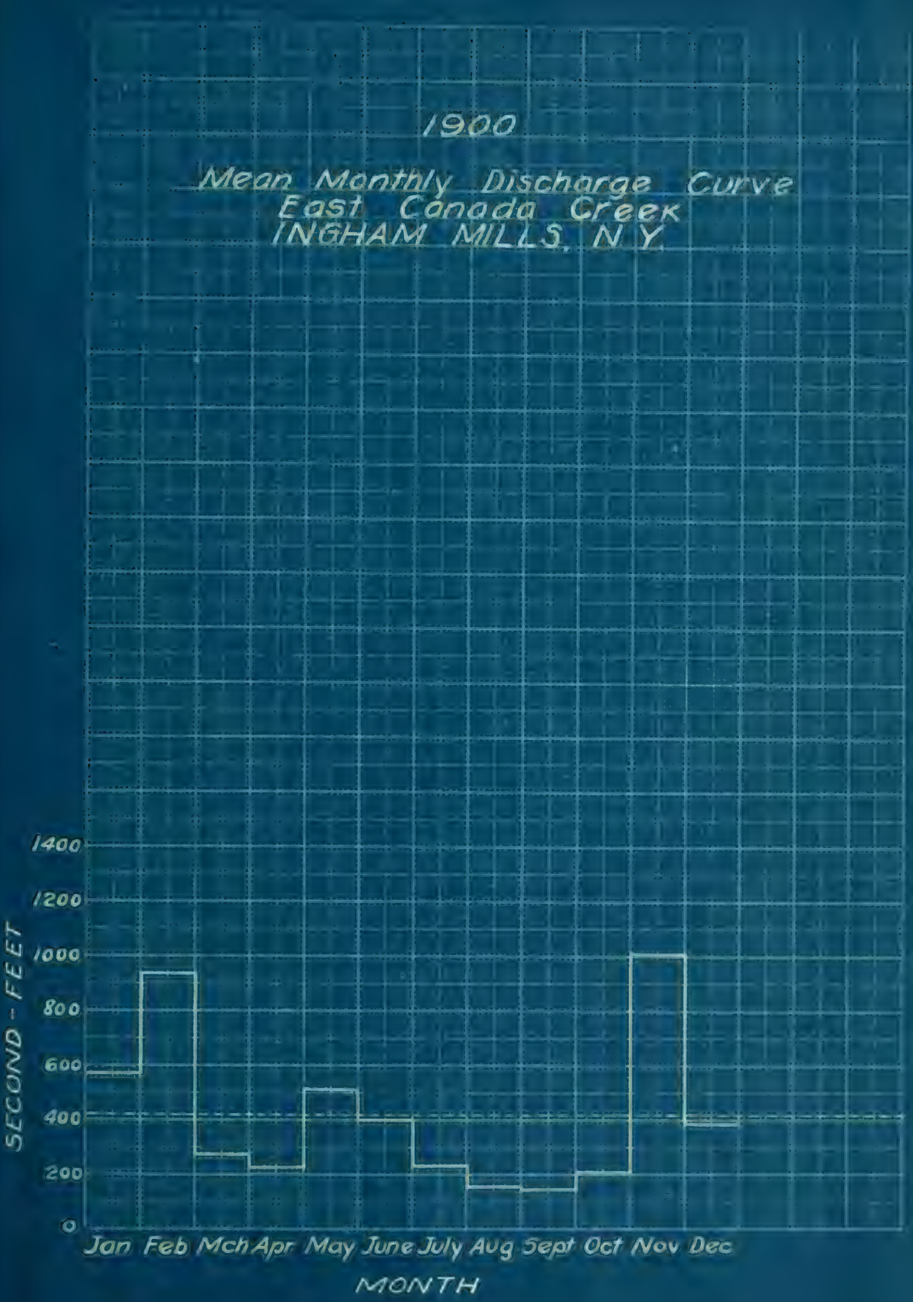
PROPOSED EAST CREEK DEVELOPMENT
INGHAM MILLS
AND
IMMEDIATE VICINITY

G W Buck. W J Keveney. G D Lettermann.
THESIS

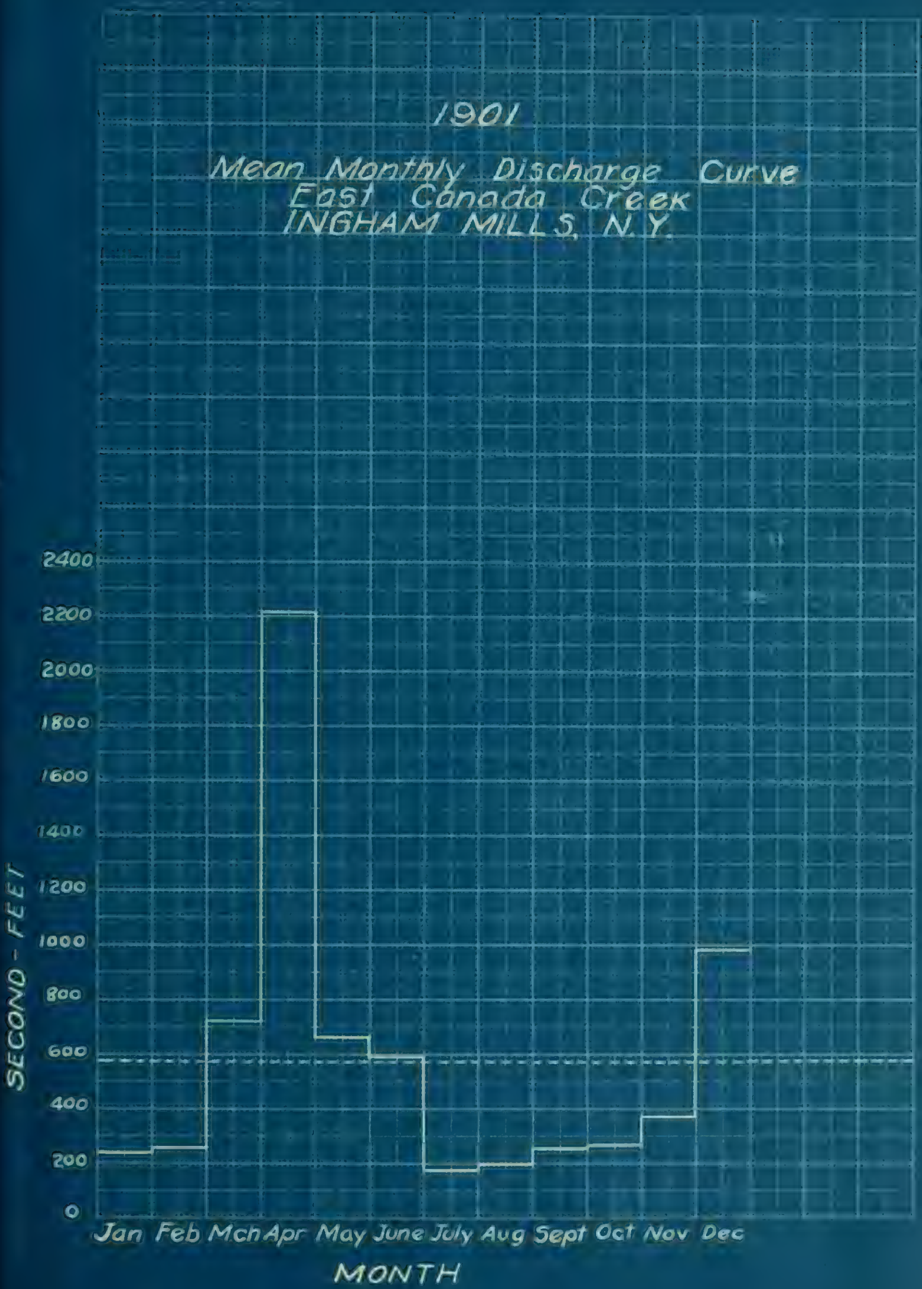
1

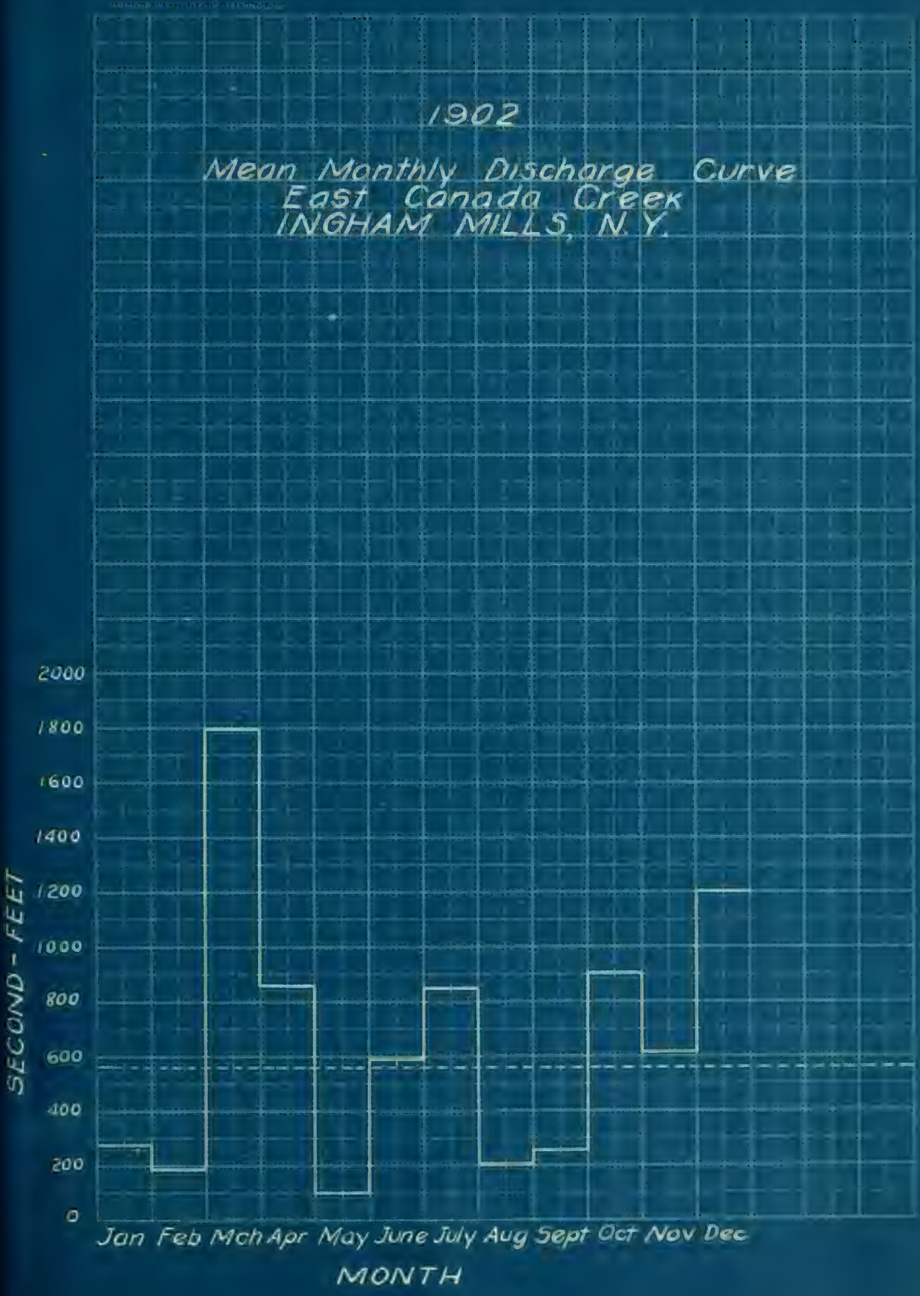

SITE OF PROPOSED PROJECT
ISLAND
EAST CANADA CREEK
East Canada Creek
General Elevation of Bottom 560
Face of Abandoned Quarry
Road to Dolgeville
Old Head Race to Mill
POWER HOUSE
El. of Water 540
El. of Water 560.50
PROPOSED DAM
PROPOSED PIPE LINE UNDER 125 FT. HEAD
SURGE TANK
Road to Johnstown
Road to St Johnsville
Road to East Creek
Road to Little Falls
Road
Barn
House
Shed
PROPOSED EAST CREEK DEVELOPMENT
AT
INGHAM MILLS, NEW YORK
THESIS
2

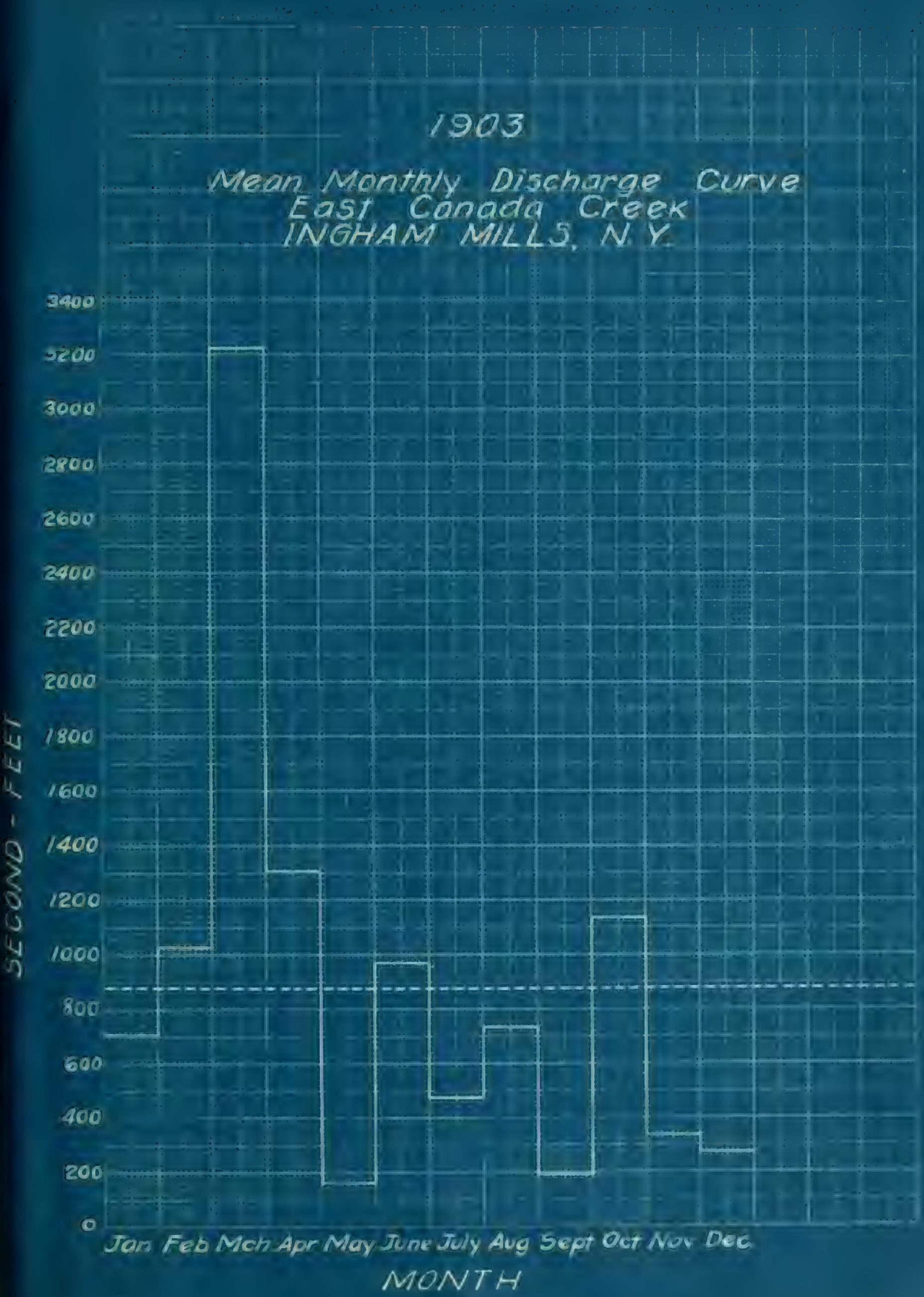

1899
Mean Monthly Discharge Curve
East Canada Creek
INGHAM MILLS N.Y.
SECOND - FEET
1000
800
600
400
200
0
Jan Feb Mch Apr May June July Aug Sept Oct Nov Dec
MONTH

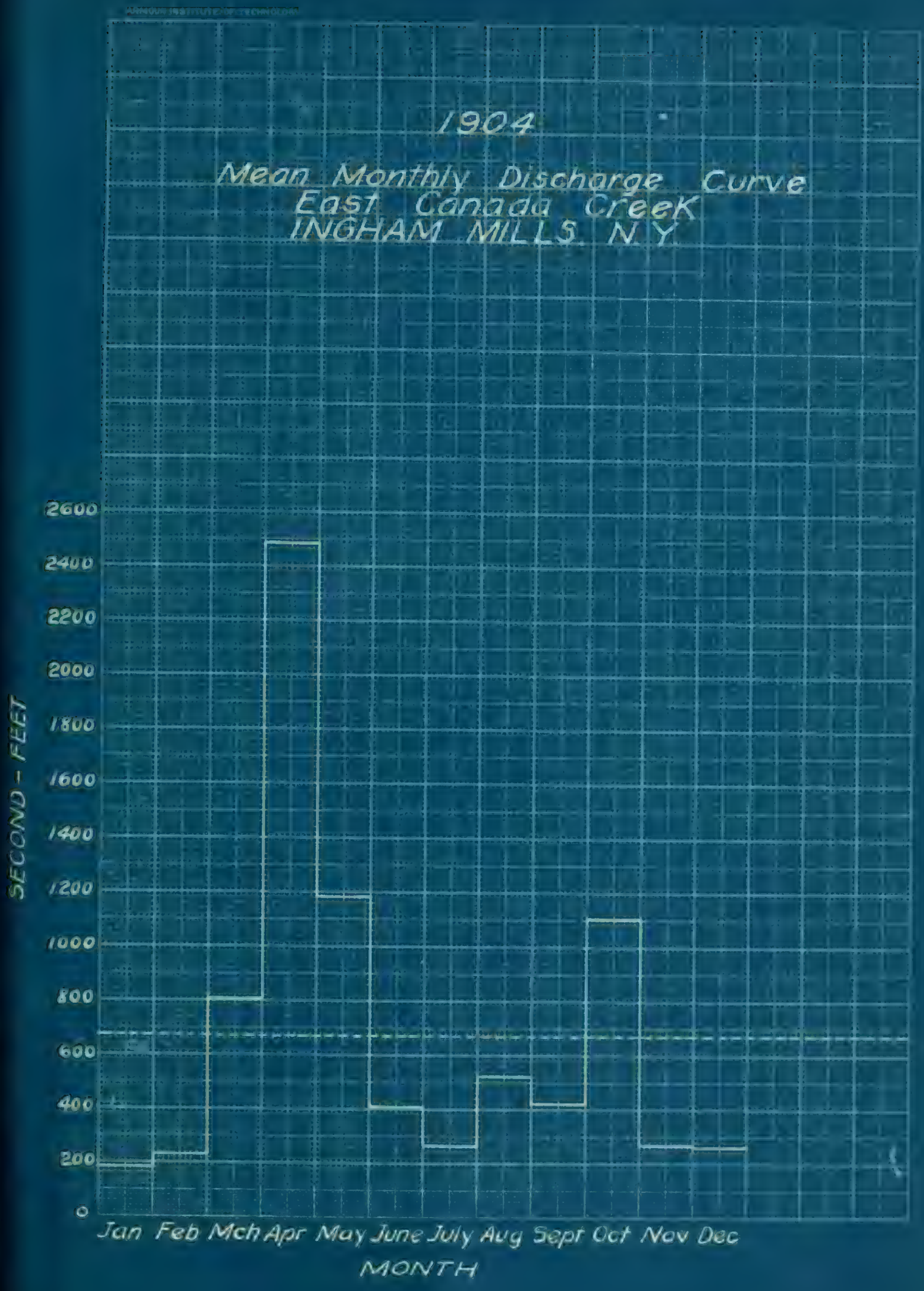


1900
Mean Monthly Discharge Curve
East Canada Creek
INGHAM MILLS, N.Y.
1400
1200
1000
800
600
400
200
0
SECOND - FEET
Jan Feb Mch Apr May June July Aug Sept Oct Nov Dec
MONTH

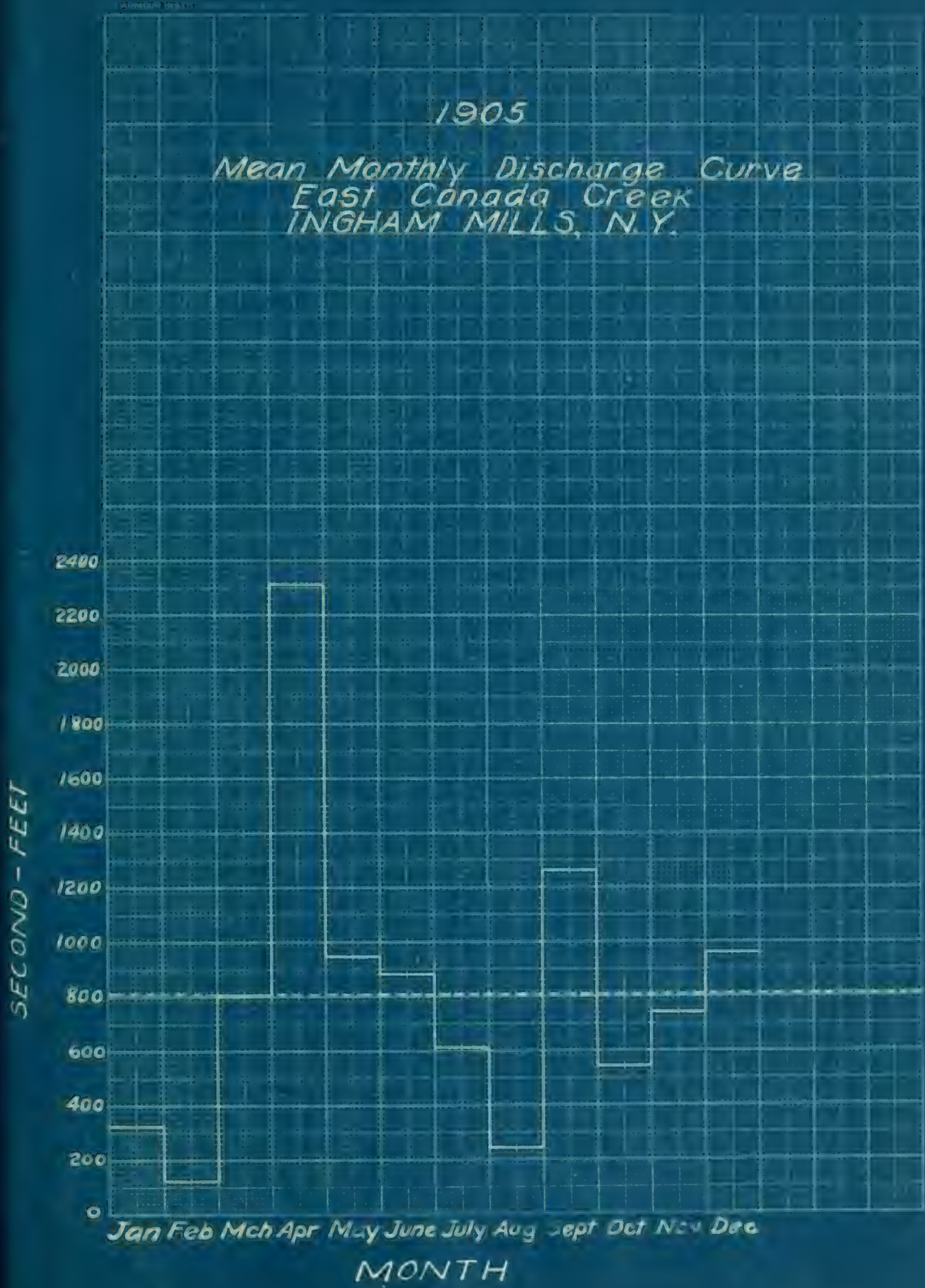

1901
Mean Monthly Discharge Curve
East Canada Creek
INGHAM MILLS, N.Y.
2400
2200
2000
1800
1600
1400
1200
1000
800
600
400
200
0
SECOND - FEET
Jan Feb Mch Apr May June July Aug Sept Oct Nov Dec
MONTH

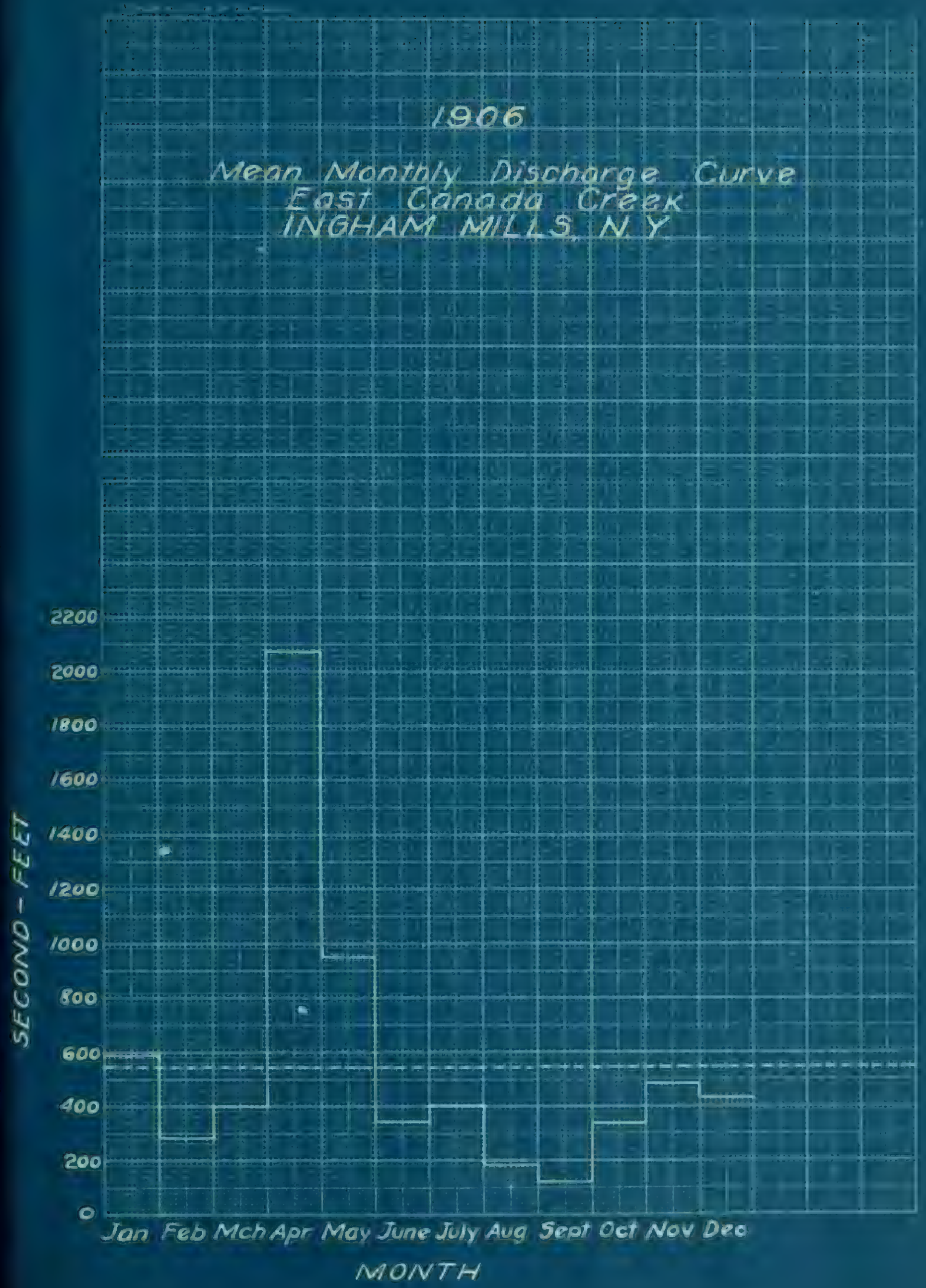

1902
Mean Monthly Discharge Curve
East Canada Creek
INGHAM MILLS, N.Y.
SECOND - FEET
2000
1800
1600
1400
1200
1000
800
600
400
200
0
Jan Feb Mch Apr May June July Aug Sept Oct Nov Dec
MONTH


1903
Mean Monthly Discharge Curve
East Canada Creek
INGHAM MILLS, N.Y.
3400
3200
3000
2800
2600
2400
2200
2000
1800
1600
1400
1200
1000
800
600
400
200
0
SECOND - FEET
Jan Feb Mch Apr May June July Aug Sept Oct Nov Dec
MONTH



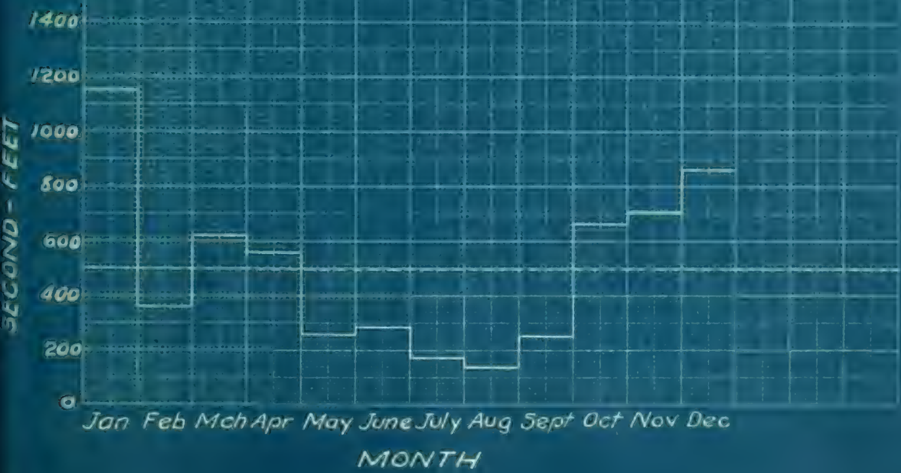
1904
Mean Monthly Discharge Curve
East Canada Creek
INGHAM MILLS. N.Y.
SECOND - FEET
2600
2400
2200
2000
1800
1600
1400
1200
1000
800
600
400
200
0
Jan Feb Mch Apr May June July Aug Sept Oct Nov Dec
MONTH

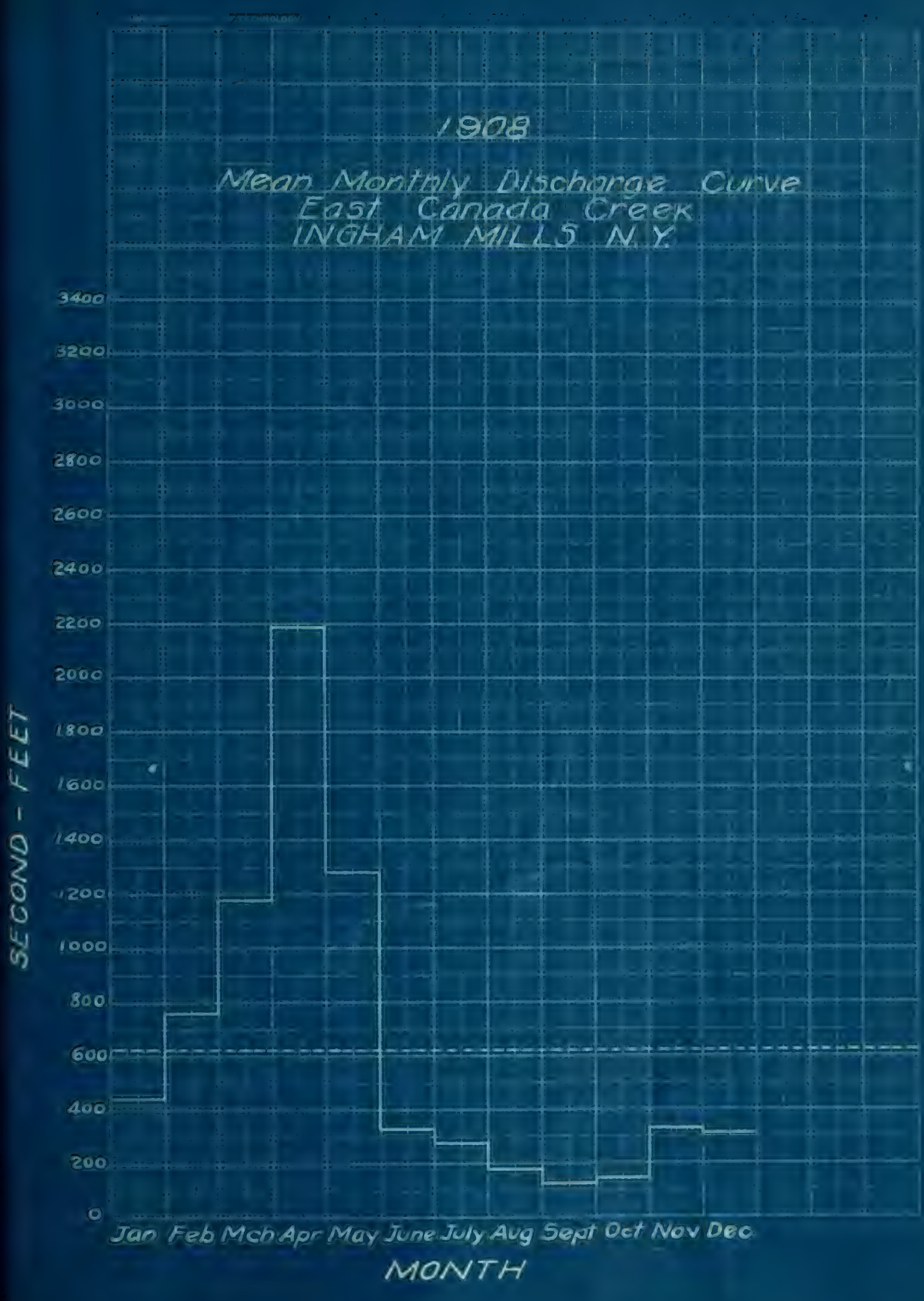

1905
Mean Monthly Discharge Curve
East Canada Creek
INGHAM MILLS, N.Y.
SECOND - FEET
0
200
400
600
800
1000
1200
1400
1600
1800
2000
2200
2400
Jan Feb Mch Apr May June July Aug Sept Oct Nov Dec
MONTH

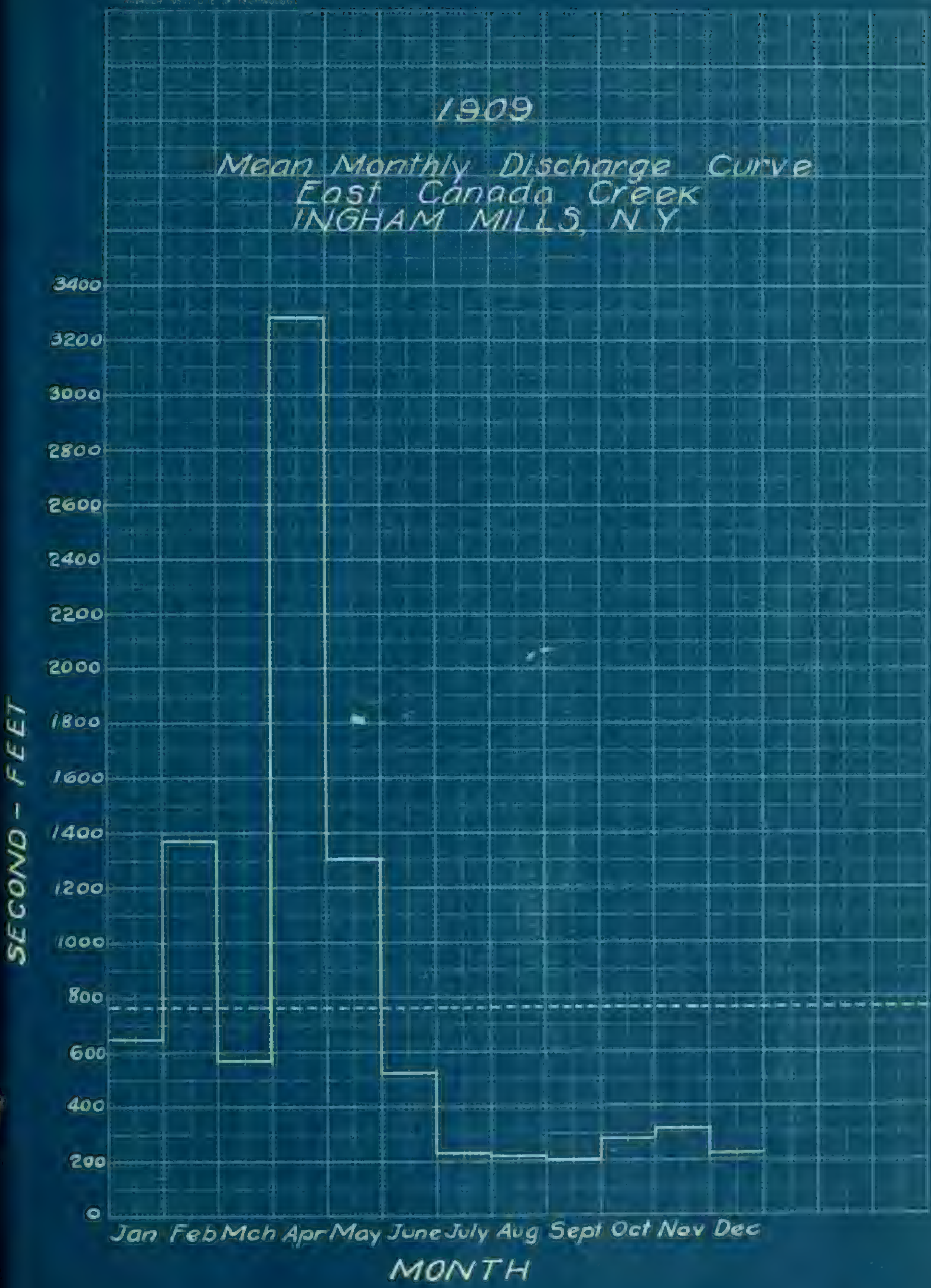

1906
Mean Monthly Discharge Curve
East Canada Creek
INGHAM MILLS, N.Y.
SECOND - FEET
2200
2000
1800
1600
1400
1200
1000
800
600
400
200
0
Jan Feb Mch Apr May June July Aug Sept Oct Nov Dec
MONTH

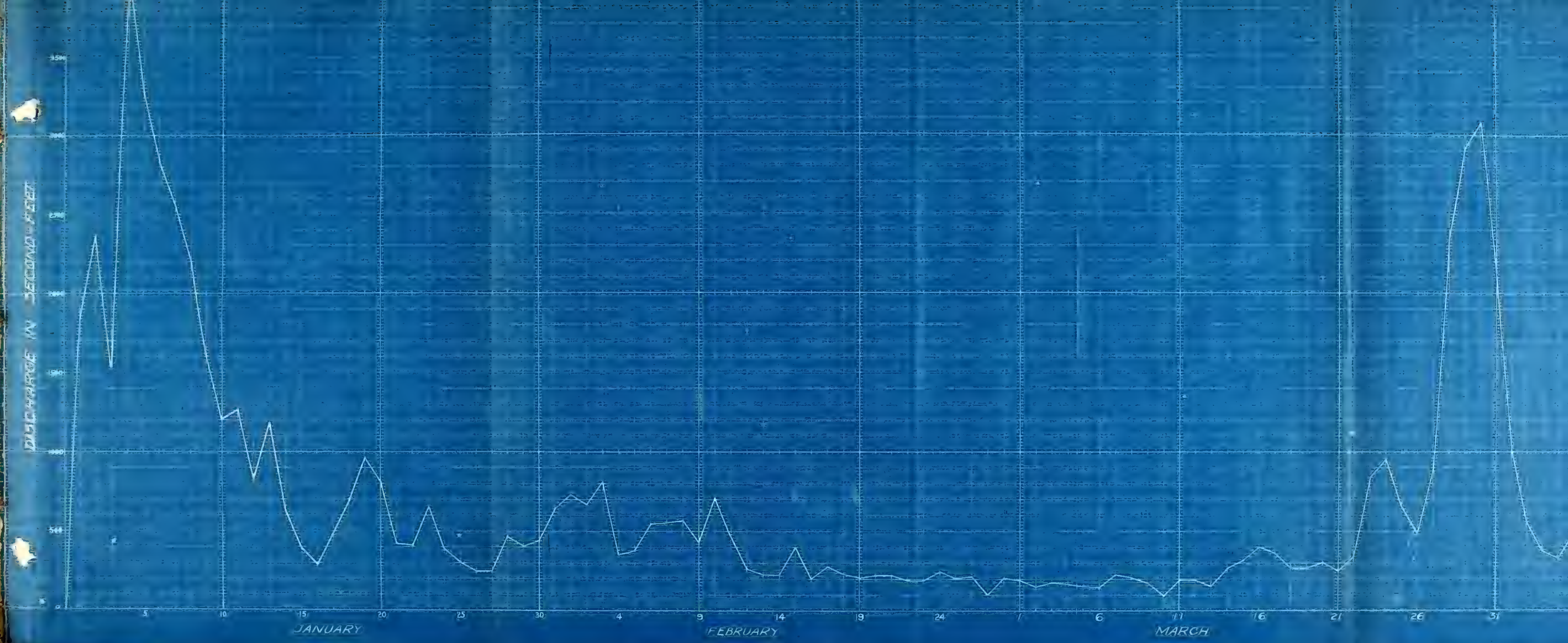
1907

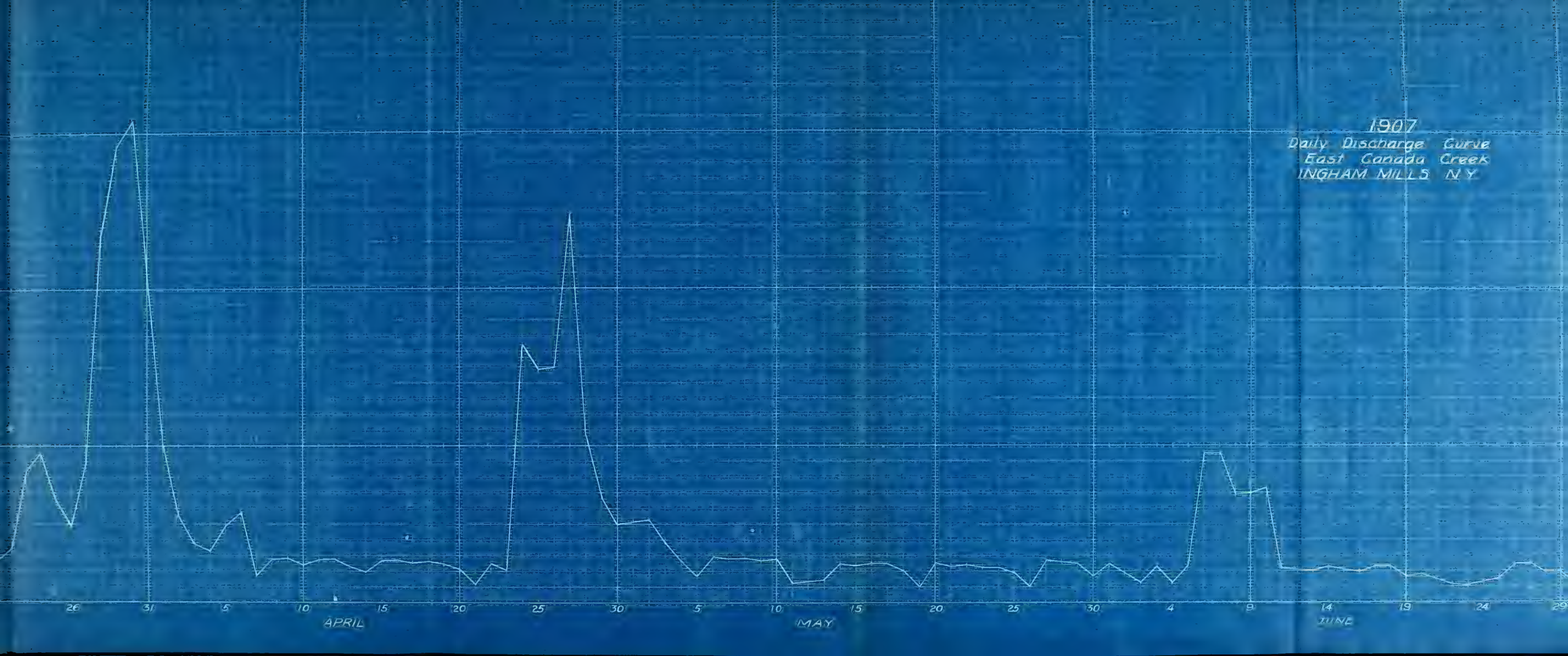
Mean Monthly Discharge Curve
East Canada Creek
INGHAM MILLS, N.Y.

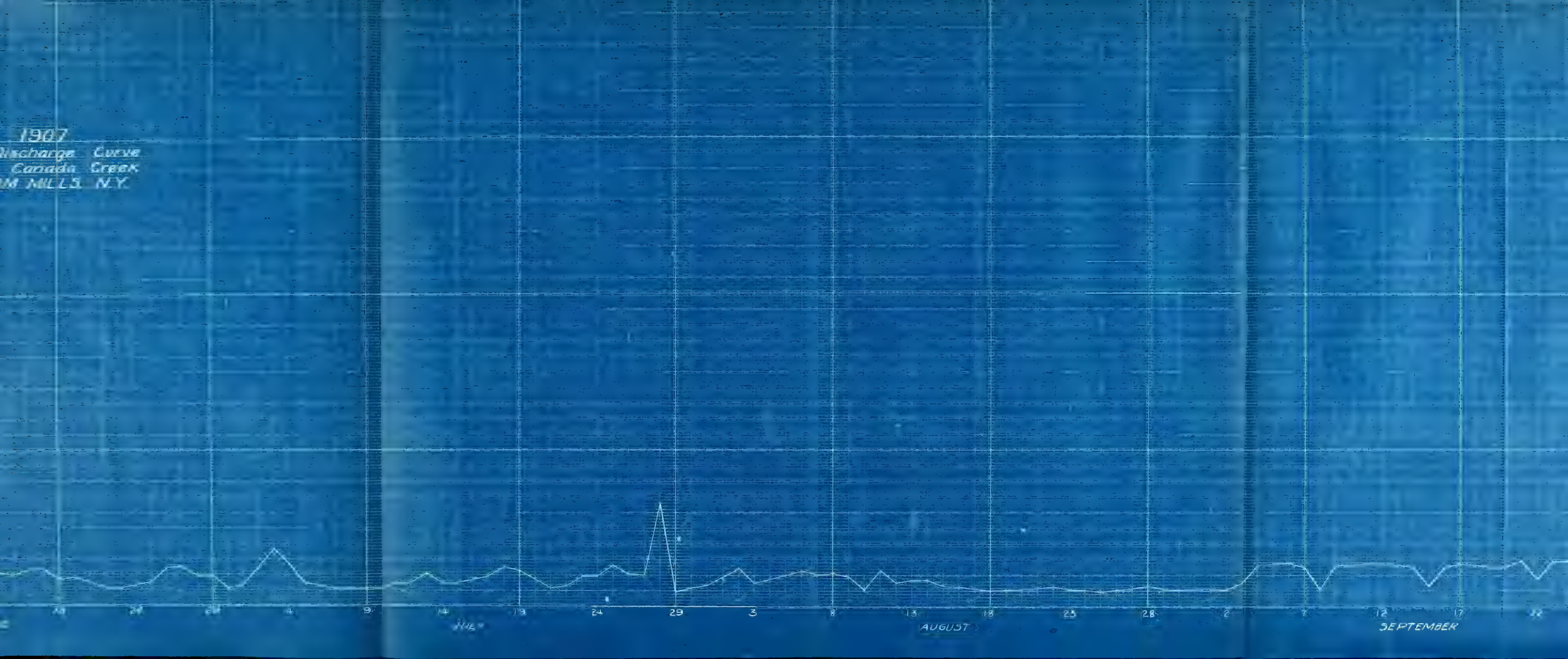


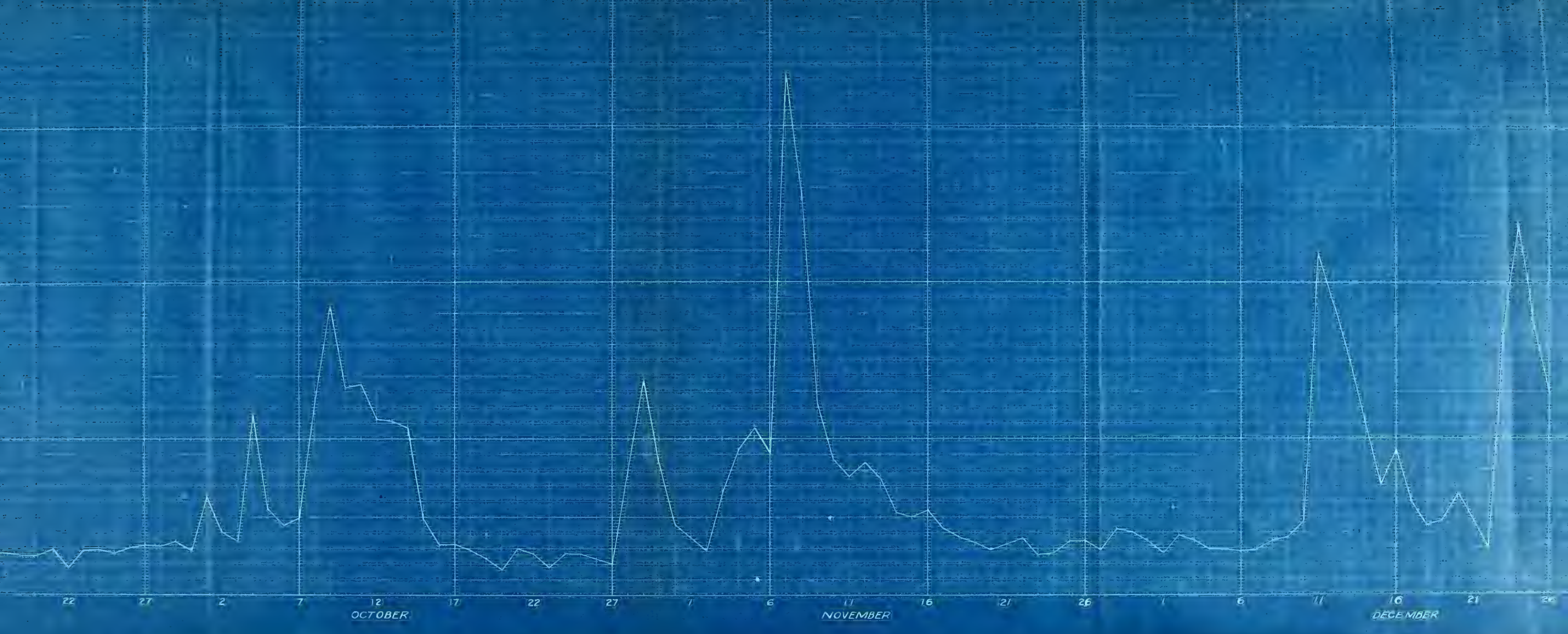

1908
Mean Monthly Discharge Curve
East Canada Creek
INGHAM MILLS N.Y.
3400
3200
3000
2800
2600
2400
2200
2000
1800
1600
1400
1200
1000
800
600
400
200
0
SECOND - FEET
Jan Feb Mch Apr May June July Aug Sept Oct Nov Dec
MONTH


1909
Mean Monthly Discharge Curve
East Canada Creek
INGHAM MILLS, N.Y.
3400
3200
3000
2800
2600
2400
2200
2000
1800
1600
1400
1200
1000
800
600
400
200
0
SECOND - FEET
Jan Feb Mch Apr May June July Aug Sept Oct Nov Dec
MONTH

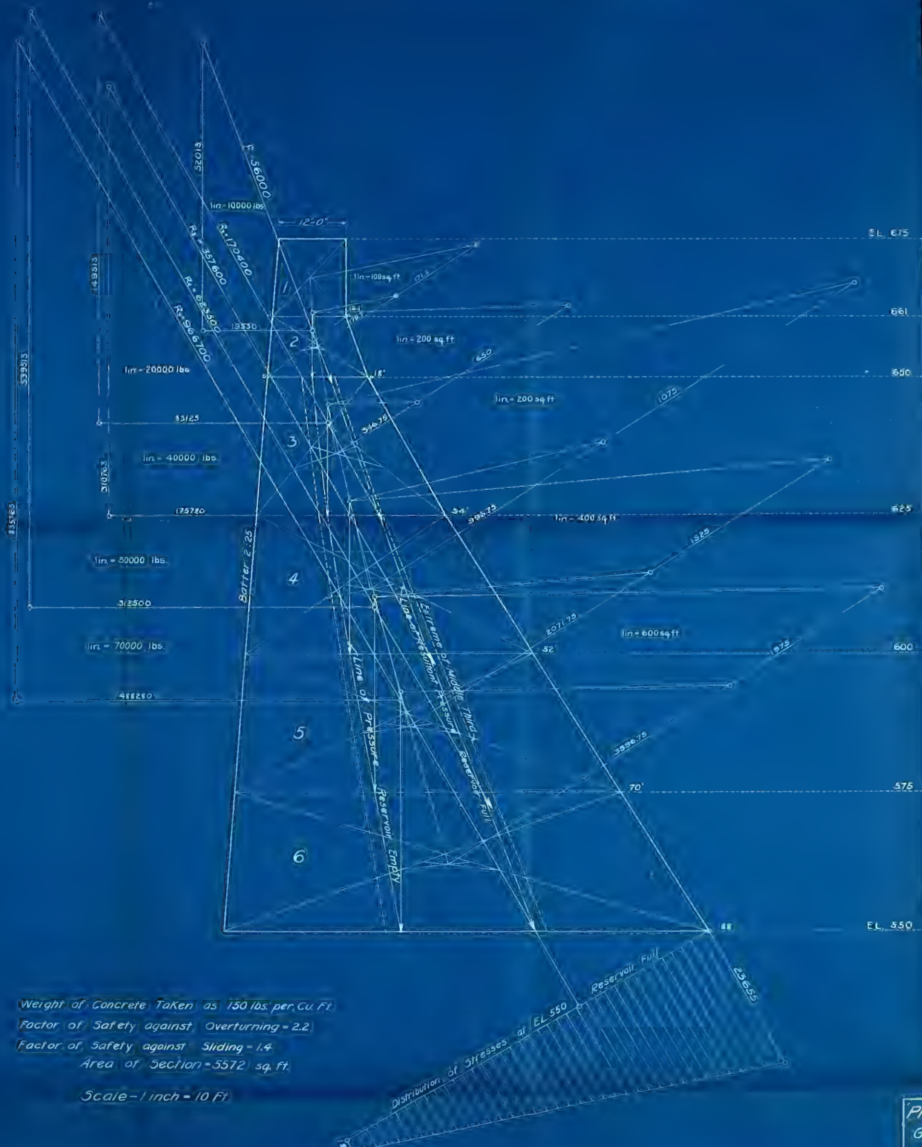

DISCHARGE IN SECOND FEET
500
5
10
15
20
25
30
4
9
14
19
24
1
6
11
16
21
26
31
JANUARY
FEBRUARY
MARCH


1907
Daily Discharge Curve
East Canada Creek
INGHAM MILLS N.Y.
26
31
5
10
15
20
25
30
5
10
15
20
25
30
4
9
14
19
24
29
APRIL
MAY
JUNE


1907
ischarge Curve
Canada Creek
M MILLS N.Y.
4
9
14
19
24
29
3
8
13
18
23
28
2
7
12
17
AUGUST
SEPTEMBER



22
27
2
7
12
OCTOBER
17
22
27
1
6
11
NOVEMBER
16
21
26
1
6
11
16
DECEMBER
21
26

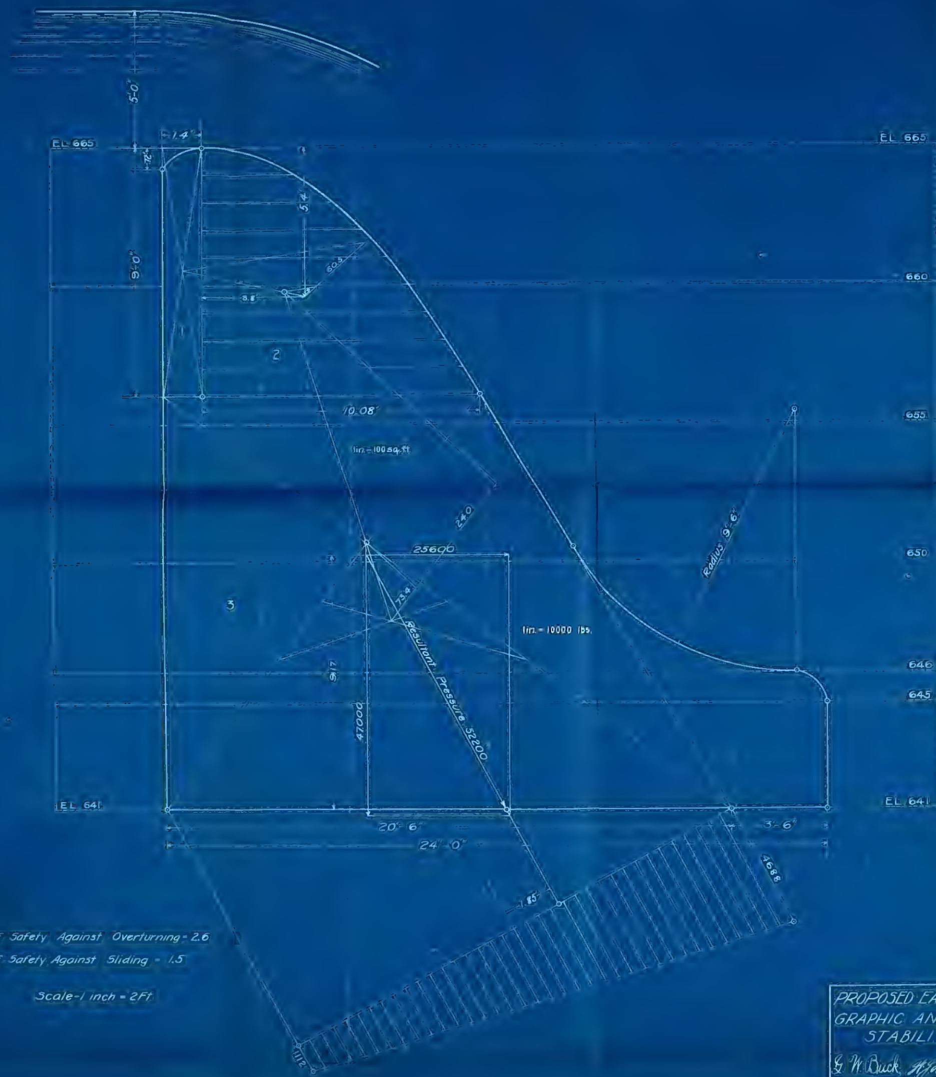


Weight of Concrete Taken as 150 lbs. per Cu. Ft.
Factor of Safety against Overturning = 2.2
Factor of Safety against Sliding = 1.4
Area of Section = 5572 sq. ft.

Scale - 1 inch = 10 Ft.

PROPOSED EAST CREEK DEVELOPMENT
GRAPHIC ANALYSIS OF DAM STABILITY AND STRESSES

G. H. Buck, W. J. Denney, G. D. Lettermann
THESIS
3



Factor of Safety Against Overturning = 2.6

Factor of Safety Against Sliding = 1.5

Scale - 1 inch = 2 Ft.

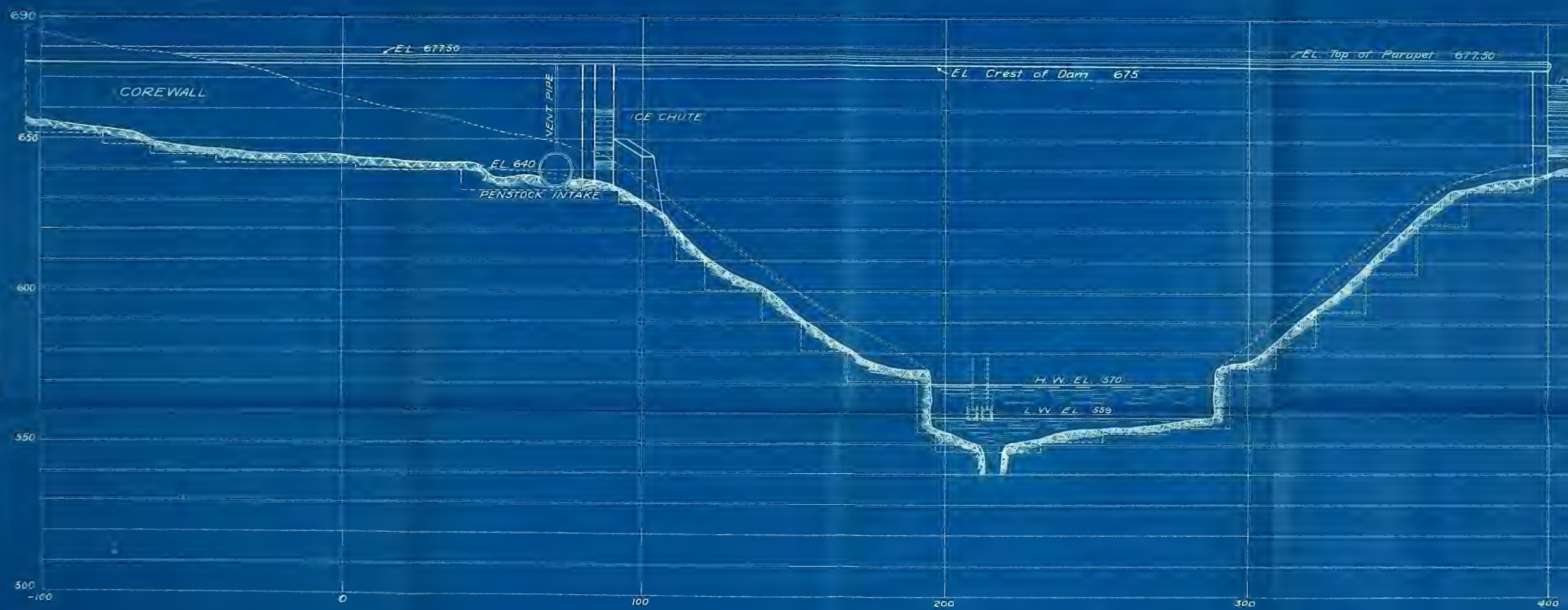
PROPOSED EAST CREEK DEVELOPMENT

GRAPHIC ANALYSIS OF SPILLWAY STABILITY AND STRESSES

G. W. Buck [illegible] G. D. Lettermann

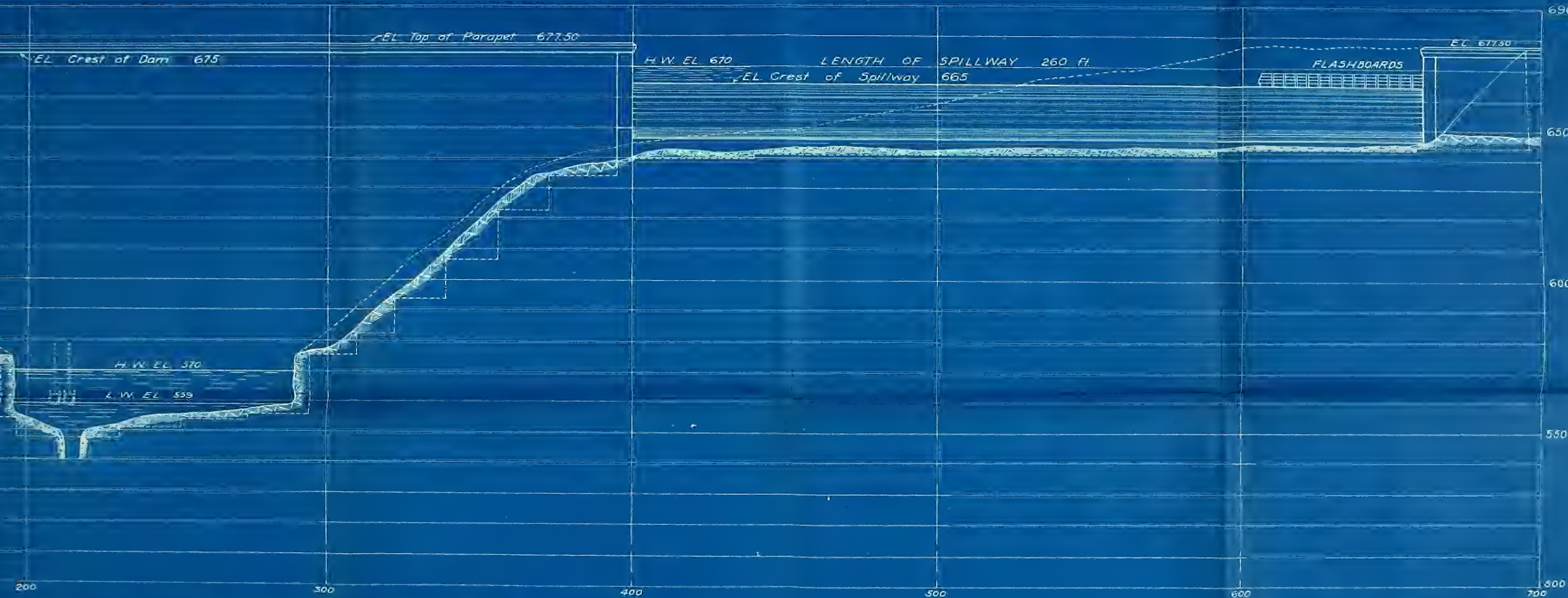
THESIS

4



DOWNSTREAM ELEVATION OF DAM AND SPILLWAY

Scale - 1 inch = 20 Ft.

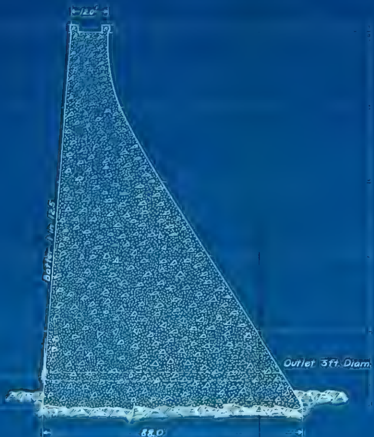


...TAM ELEVATION OF DAM AND SPILLWAY

Scale - 1 inch = 20 Ft

PROPOSED EAST CREEK DEVELOPMENT
AT
INGHAM MILLS, NEW YORK
G. N. Zuck, H. J. Kenney & G. D. Lettermann
THESIS

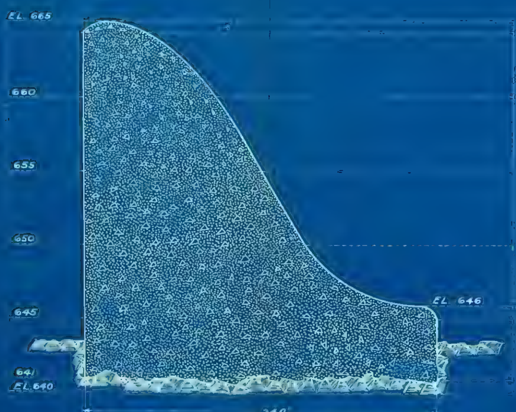
5



CROSS SECTION OF DAM

Scale-1 inch-20 ft.

CROSS-SECTION THRU INTAKE



CROSS SECTION THRU SPILLWAY

Scale-1 inch-4 ft.

FLASHBOARDS ARRANGEMENT

Scale: 1 inch = 1 Foot

CROSS-SECTION THRU COREWALL

PROPOSED EAST CREEK DEVELOPMENT

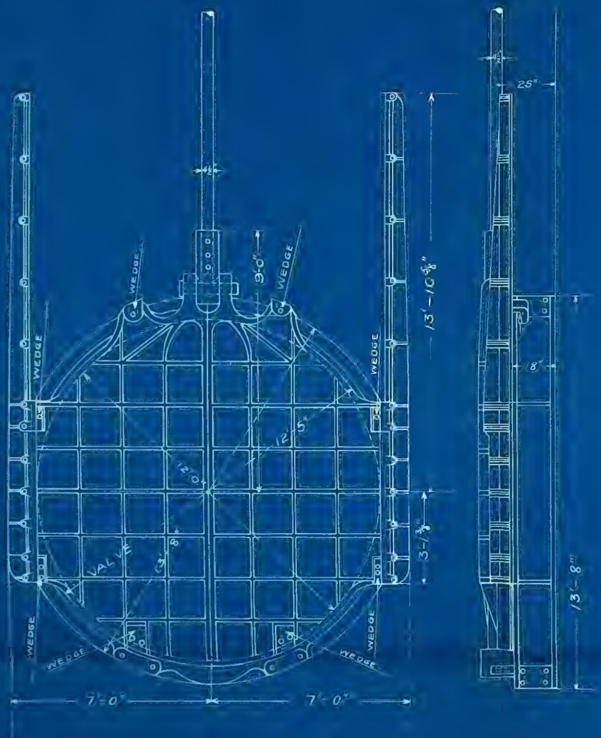
AT

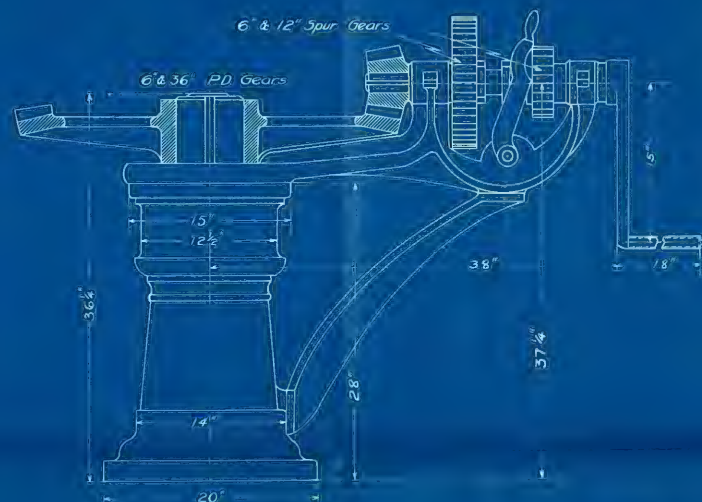
INGHAM MILLS, NEW YORK

G.W. Buck; H.J. [illegible]; G.L. Zettermann

THESIS

6





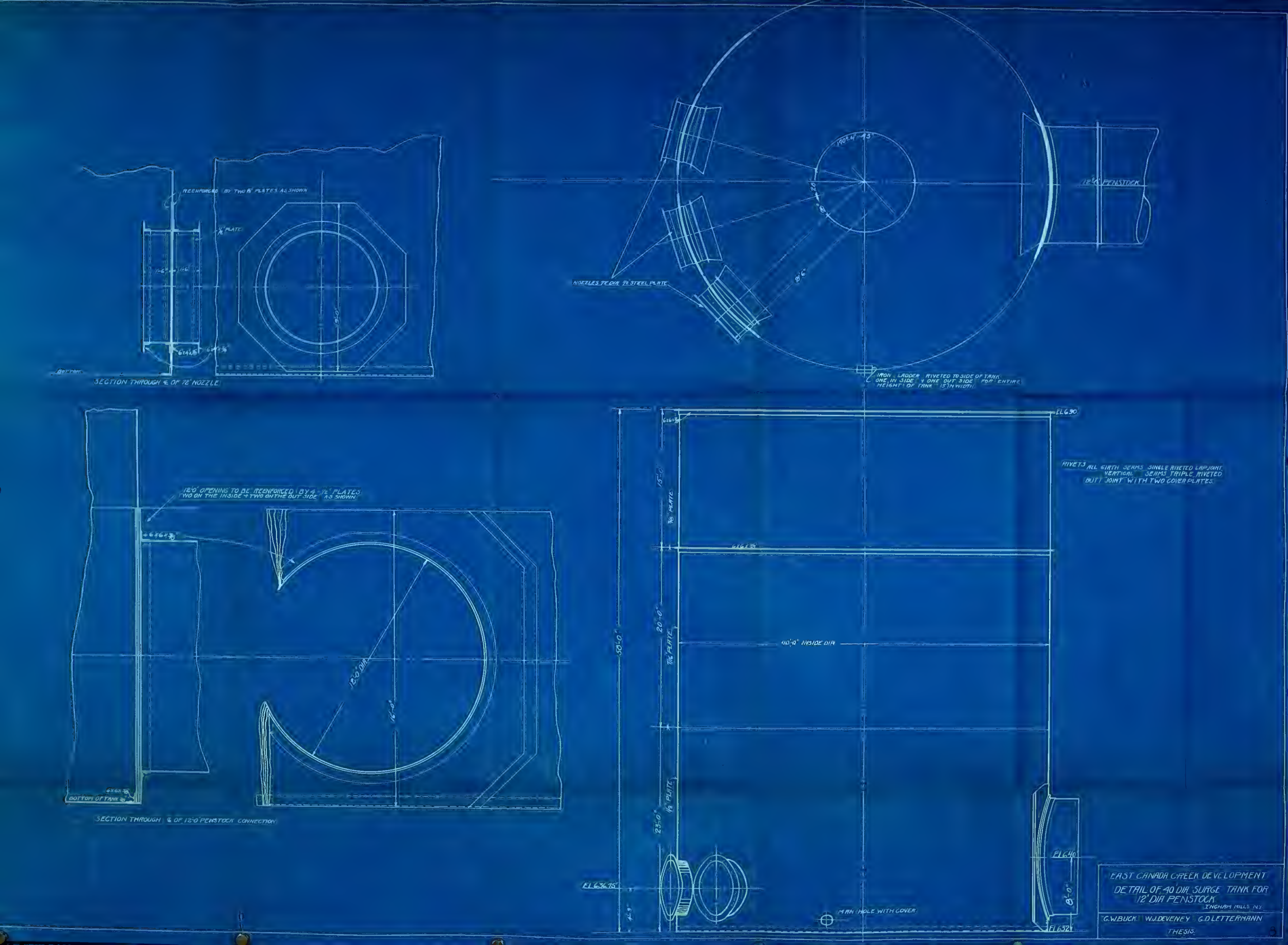
Scale - 1½ inch = 1 Ft.

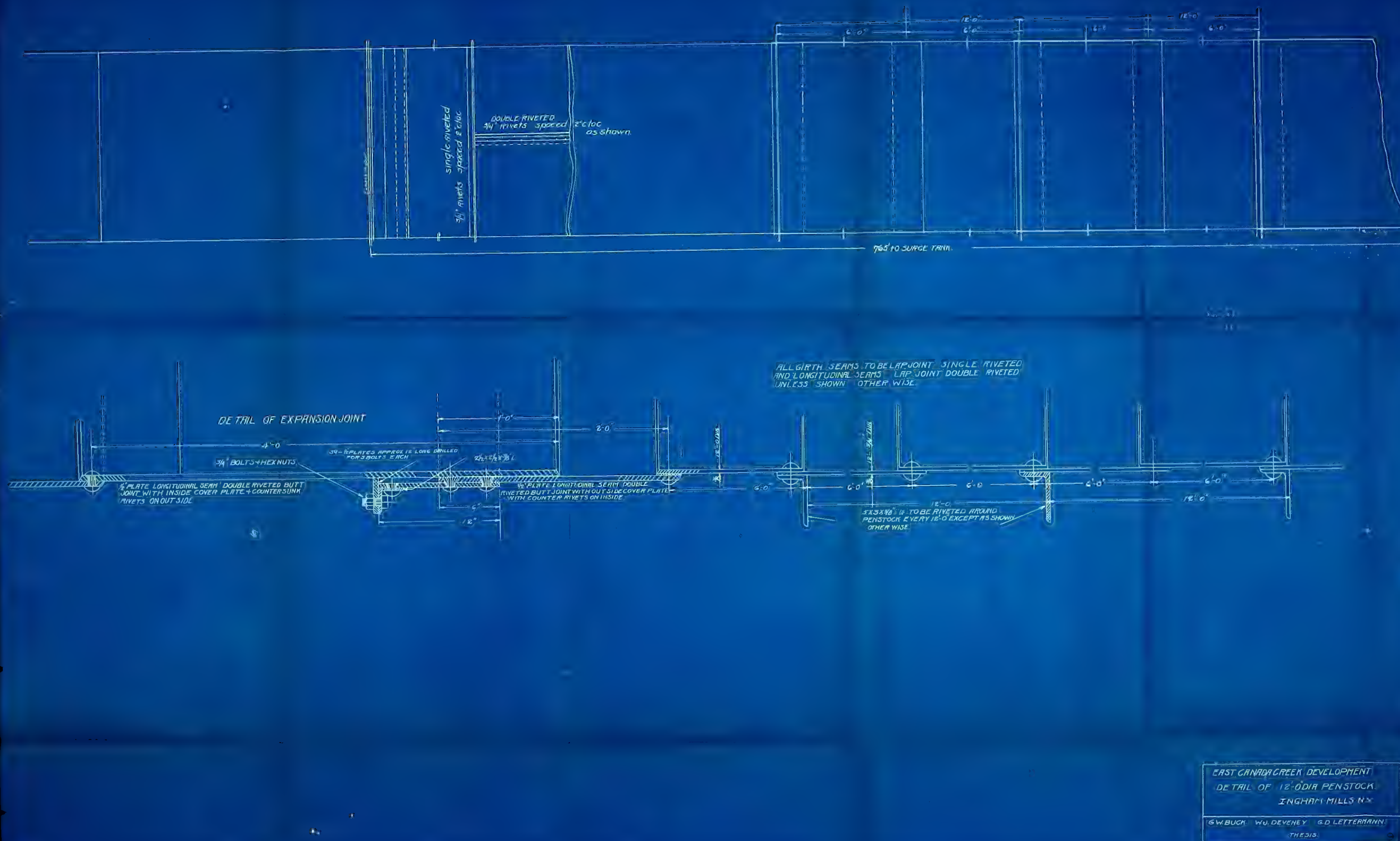
PROPOSED EAST CREEK DEVELOPMENT
SLUICE GATE
AND
OPERATING STAND

G. W. Buck. H. J. Kenney. G. D. Lettermann

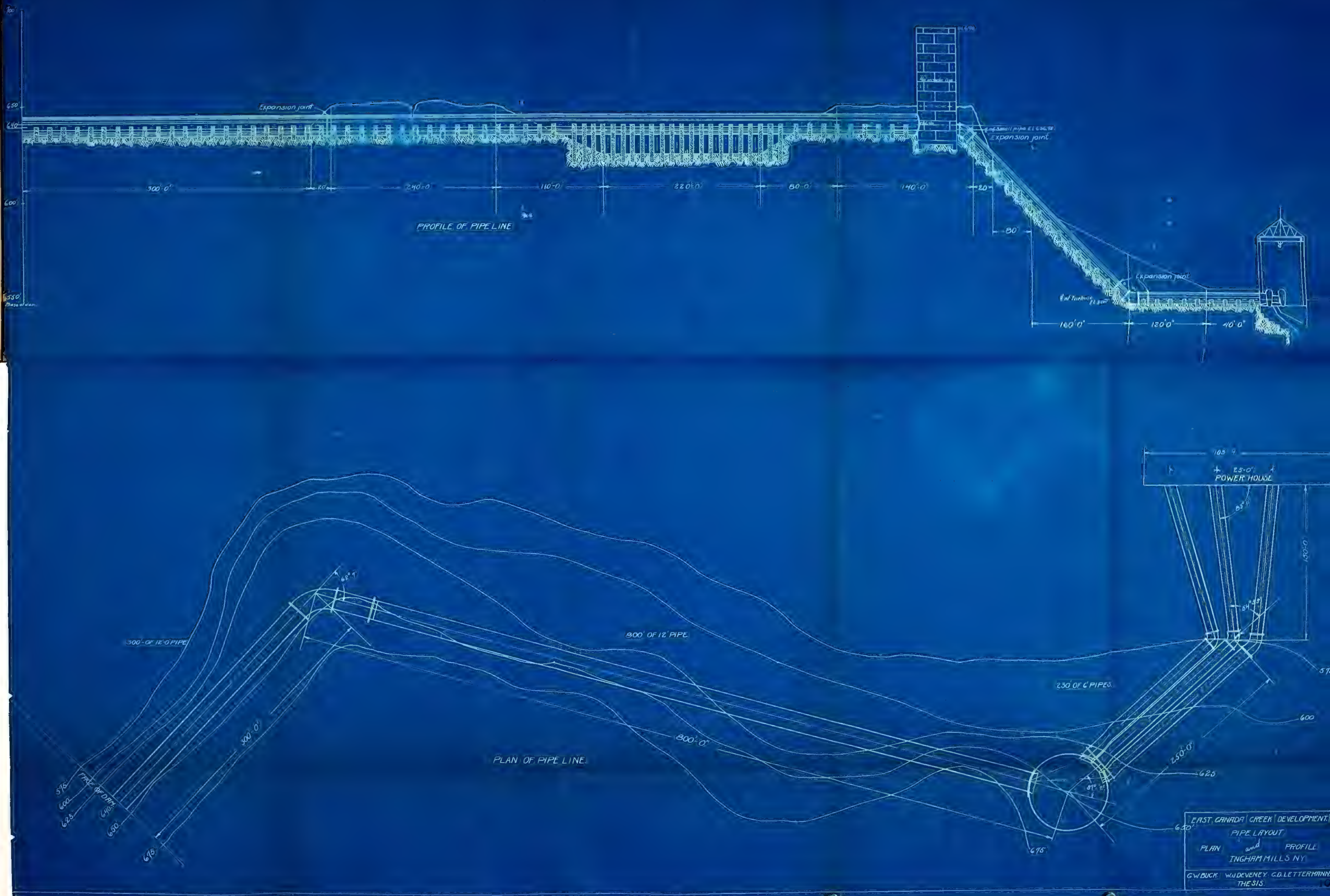
THESIS

7

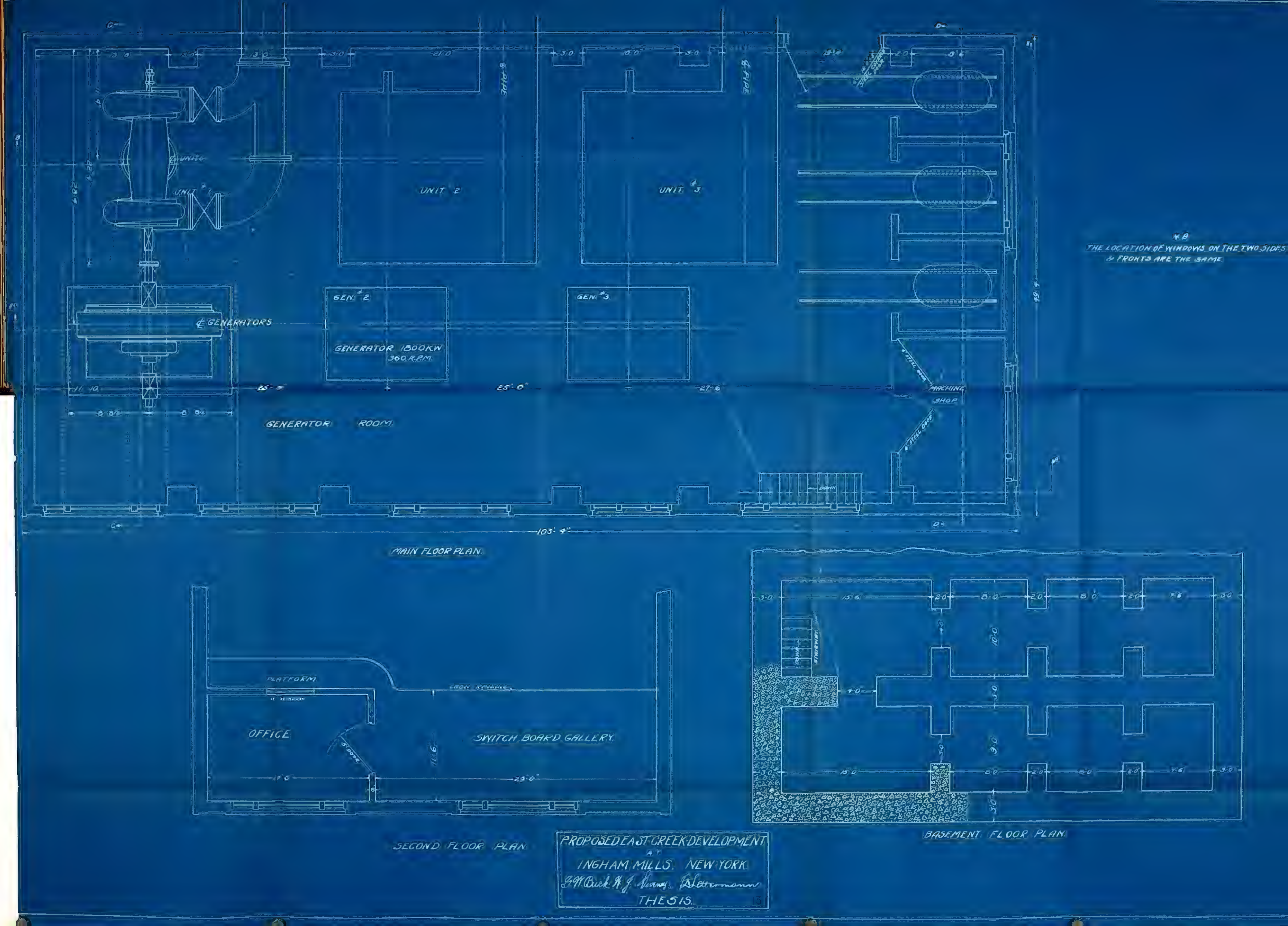

REENFORCED BY TWO ½" PLATES AS SHOWN
½" PLATE
11'-6"
6x4x⅝
6x4x⅝
BOTTOM
SECTION THROUGH ℄ OF 72" NOZZLE
NOZZLES 72" DIA ½" STEEL PLATE
12'Ø PENSTOCK
IRON LADDER RIVETED TO SIDE OF TANK ONE IN SIDE + ONE OUT SIDE FOR ENTIRE HEIGHT OF TANK 15" IN WIDTH
EL 690
RIVETS ALL GIRTH SEAMS SINGLE RIVETED LAP JOINT
VERTICAL SEAMS TRIPLE RIVETED BUTT JOINT WITH TWO COVER PLATES
12'-0" OPENING TO BE REENFORCED BY 4 - ½" PLATES TWO ON THE INSIDE + TWO ON THE OUT SIDE AS SHOWN
6x6x⅝
12'-0" DIA
16'-0"
BOTTOM OF TANK
SECTION THROUGH ℄ OF 12'-0" PENSTOCK CONNECTION
15'-0"
⅜" PLATE
20'-0"
7/16" PLATE
40'-0" INSIDE DIA
23'-0"
½" PLATE
58'-0"
EL 636.75
EL 640
8'-0"
EL 632
MAN HOLE WITH COVER
EAST CANADA CREEK DEVELOPMENT
DETAIL OF 40' DIA SURGE TANK FOR 12' DIA PENSTOCK
INGHAM MILLS N.Y.
G.W.BUCK. W.J.DEVENEY G.D.LETTERMANN
THESIS.
8

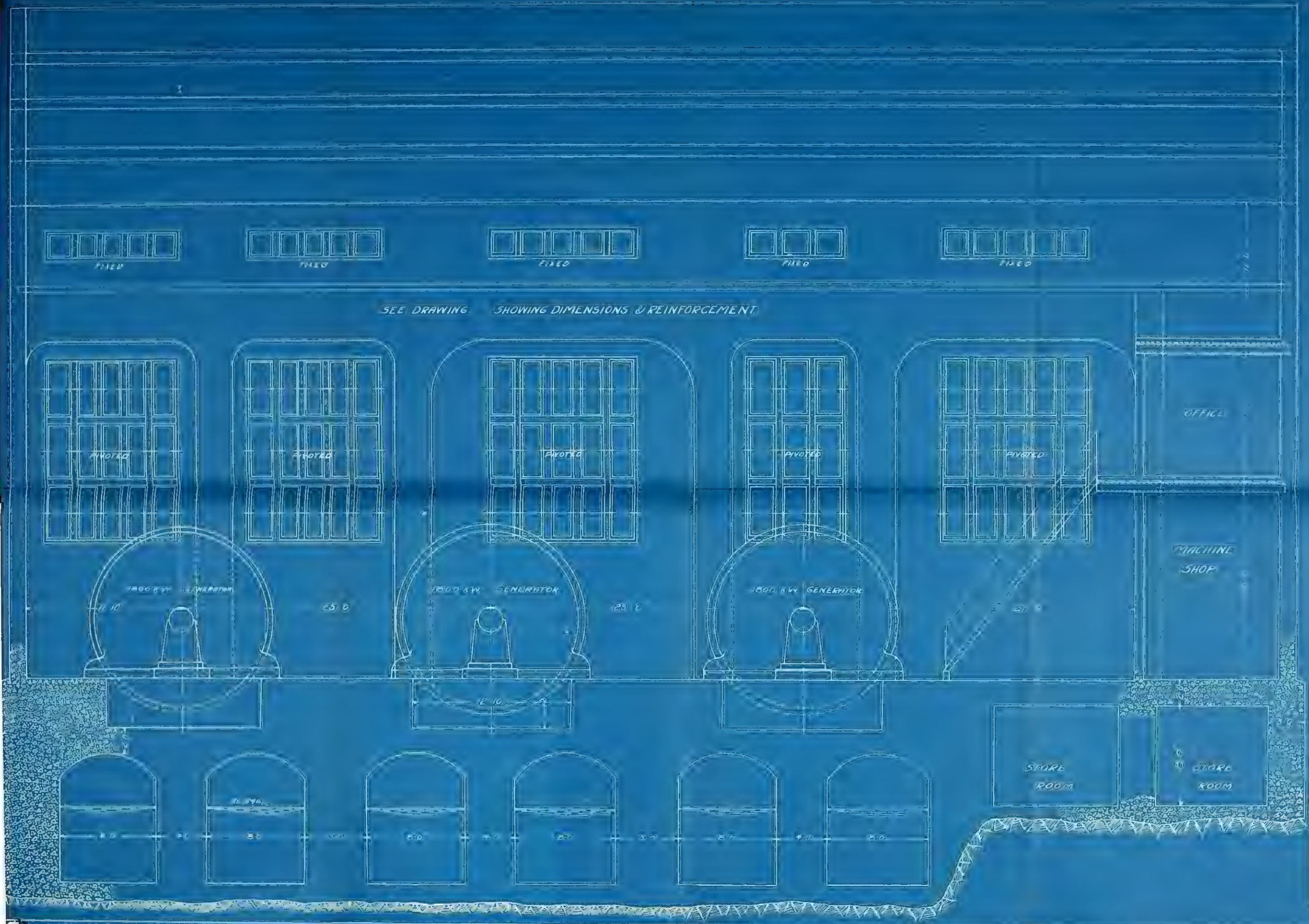


single riveted
3/4" rivets spaced 2" c/oc
DOUBLE RIVETED
3/4" rivets spaced 2" c/oc
as shown
765' to SURGE TANK
ALL GIRTH SEAMS TO BE LAP JOINT SINGLE RIVETED AND LONGITUDINAL SEAMS LAP JOINT DOUBLE RIVETED UNLESS SHOWN OTHER WISE.
DETAIL OF EXPANSION JOINT
3/4" BOLTS + HEX NUTS
39 - ½" PLATES APPROX 12" LONG DRILLED FOR 3 BOLTS EACH
½" PLATE LONGITUDINAL SEAM DOUBLE RIVETED BUTT JOINT WITH INSIDE COVER PLATE + COUNTERSUNK RIVETS ON OUT SIDE
½" PLATE LONGITUDINAL SEAM DOUBLE RIVETED BUTT JOINT WITH OUTSIDE COVER PLATE WITH COUNTER RIVETS ON INSIDE
5"X3"X⅜" L'S TO BE RIVETED AROUND PENSTOCK EVERY 12'-0" EXCEPT AS SHOWN OTHER WISE.
EAST CANADA CREEK DEVELOPMENT
DETAIL OF 12'-0" DIA PENSTOCK.
INGHAM MILLS N.Y.
G.W. BUCK W.J. DEVENEY G.D. LETTERMANN.
THESIS.
9



Expansion joint
300'-0"
240'-0"
110'-0"
220'-0"
80'-0"
140'-0"
PROFILE OF PIPE LINE
Expansion joint
160'-0"
120'-0"
40'-0"
POWER HOUSE
25'-0"
300' OF 12'-0 PIPE
800' OF 12' PIPE
230' OF 6 PIPES
300'-0"
800'-0"
250'-0"
PLAN OF PIPE LINE
FACE OF DAM
575
600
625
650
675
EAST CANADA CREEK DEVELOPMENT.
PIPE LAYOUT
PLAN and PROFILE
INGHAM MILLS NY
G.W.BUCK. W.J.DEVENEY. G.D.LETTERMANN.
THESIS.

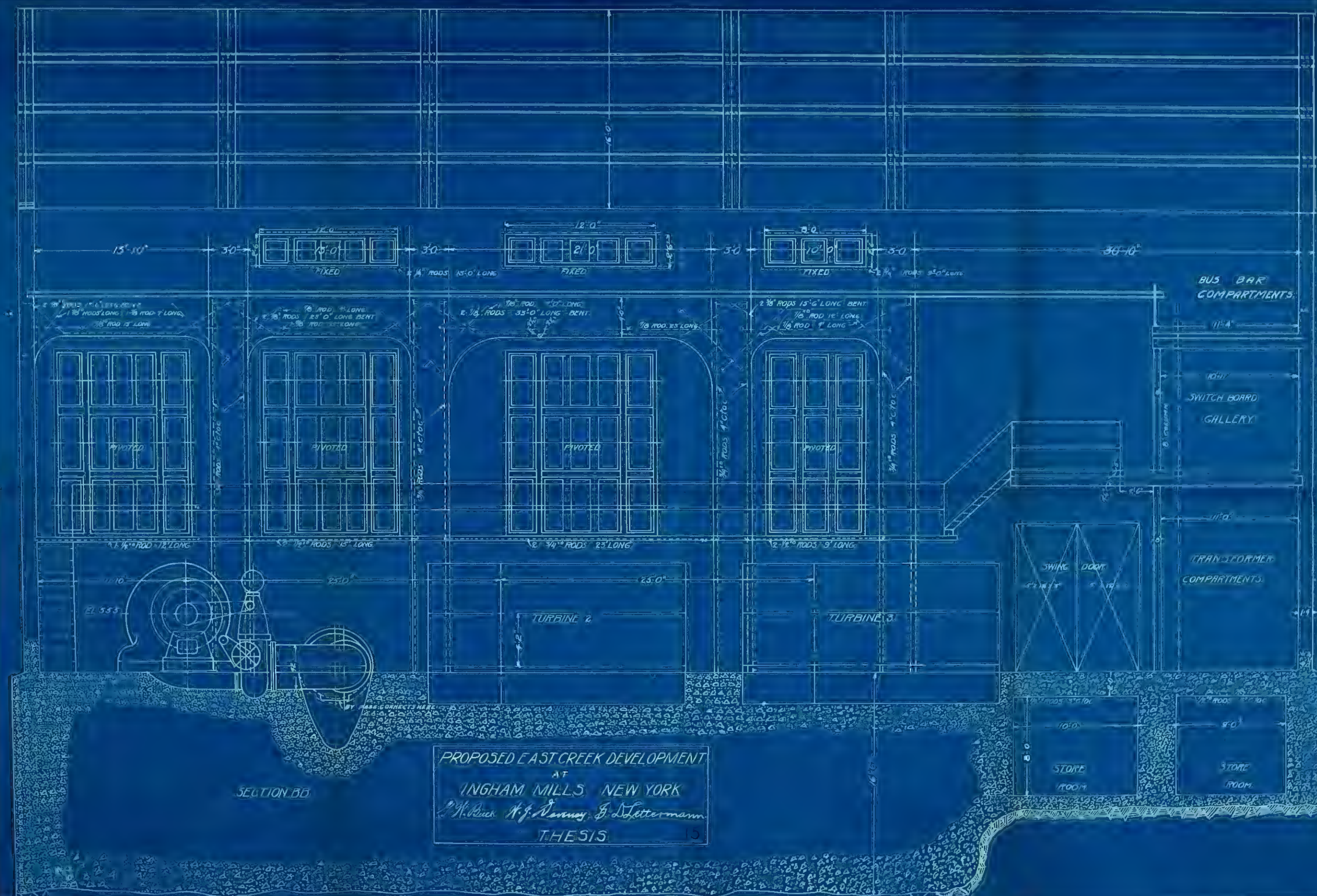

UNIT 2
UNIT #3
GEN. #2
GEN. #3
GENERATOR 1800KW 360 R.P.M.
℄ GENERATORS
GENERATOR ROOM
MACHINE SHOP
DOWN
8 PIPE
N.B.
THE LOCATION OF WINDOWS ON THE TWO SIDES & FRONTS ARE THE SAME.
103'-4"
MAIN FLOOR PLAN.
PLATFORM
IRON RAILING
OFFICE
SWITCH BOARD GALLERY.
3' DOOR
STAIRWAY
SECOND FLOOR PLAN.
BASEMENT FLOOR PLAN.
PROPOSED EAST CREEK DEVELOPMENT
AT
INGHAM MILLS, NEW YORK.
THESIS.

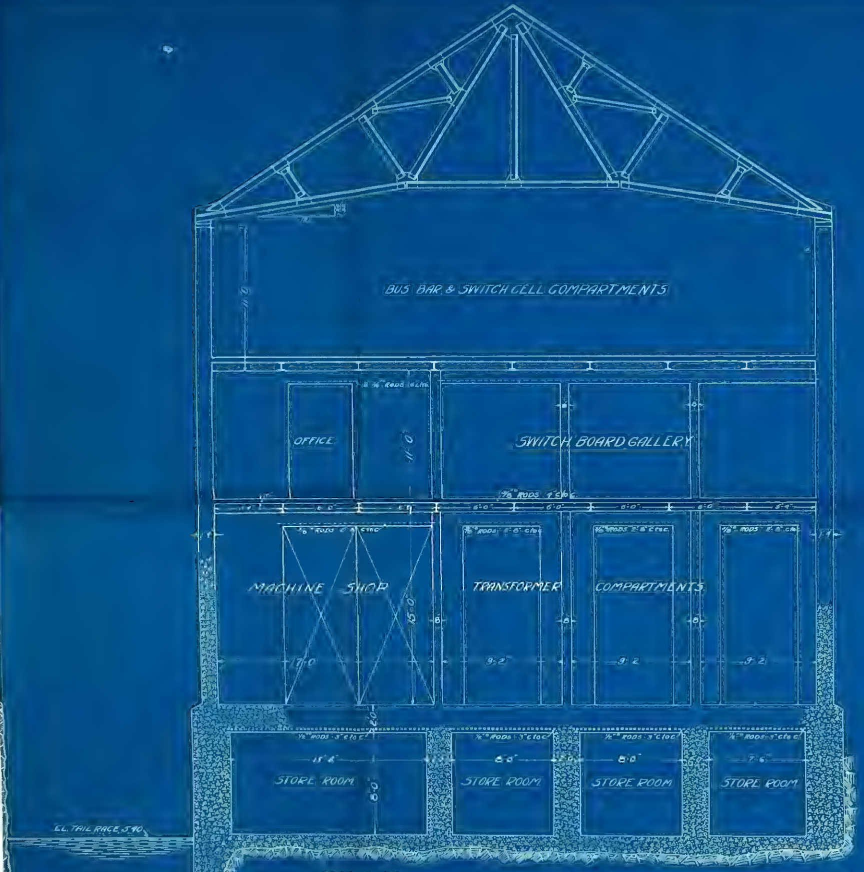


PROPOSED EAST-CREEK DEVELOPMENT
AT
INGHAM MILLS NEW YORK

THESIS

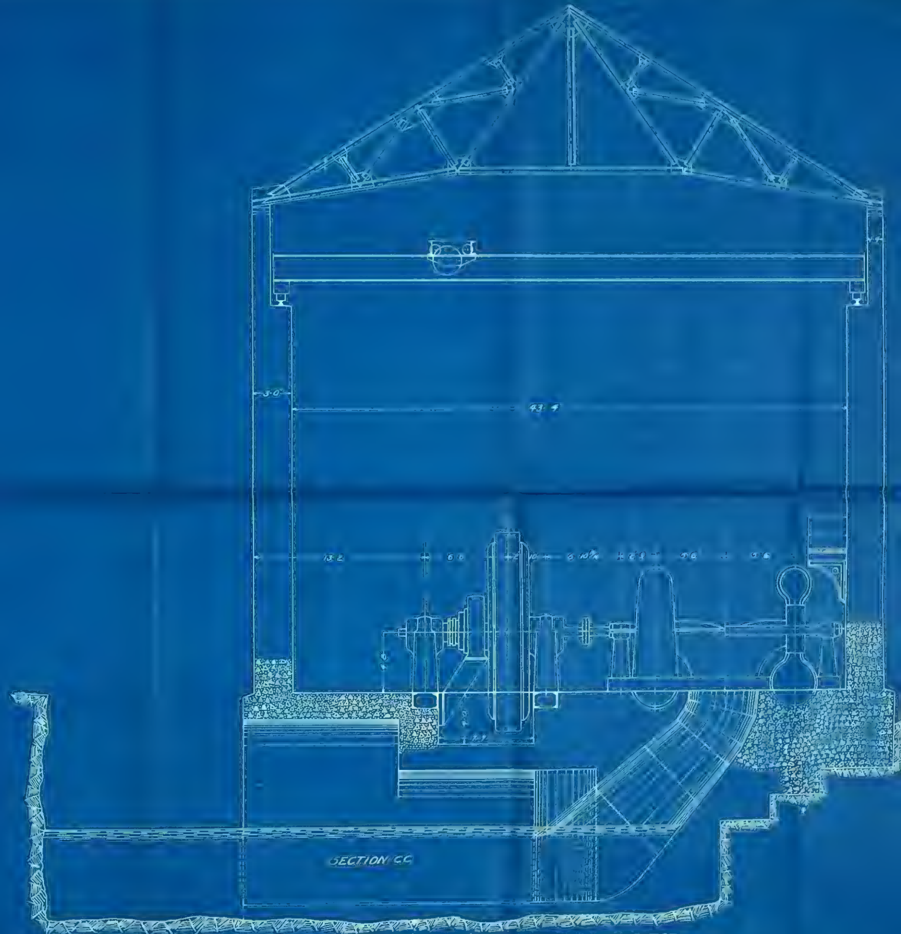
14


PROPOSED EAST CREEK DEVELOPMENT
AT
INGHAM MILLS NEW YORK
THESIS
15
SECTION BB
FIXED
PIVOTED
TURBINE 2
TURBINE 3
BUS BAR COMPARTMENTS
SWITCH BOARD GALLERY
TRANSFORMER COMPARTMENTS
SWING DOOR
STORE ROOM
STORE ROOM
15'-10"
3'-0"
12'-0"
21'-0"
10'-0"
30'-10"
25'-0"
EL 555

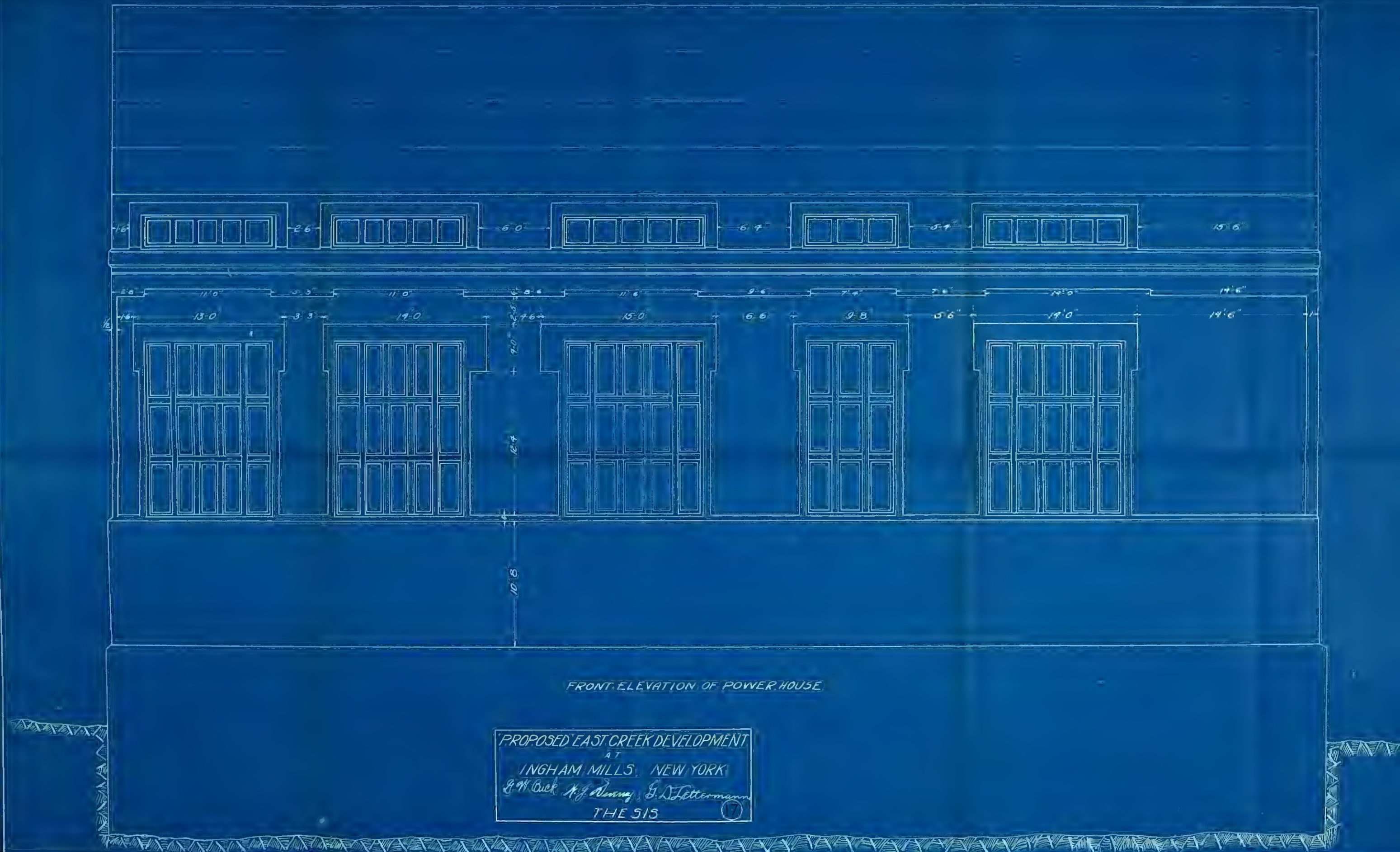


SECTION DD

PROPOSED EAST CREEK DEVELOPMENT
AT
INGHAM MILLS NEW YORK
G. W. Buck, H. J. Devenny, G. D. Lettermann
THESIS
16

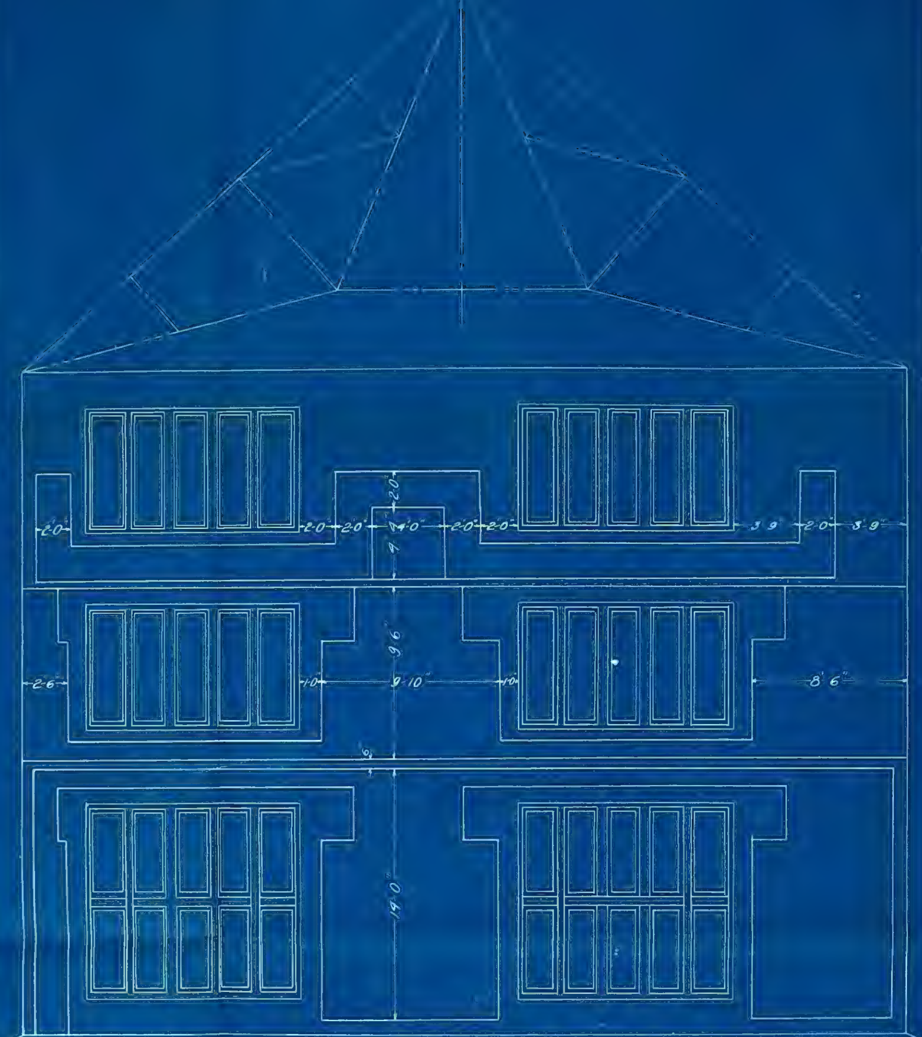


SECTION CC THROUGH ℄ OF TURBINE & GENERATOR



FRONT ELEVATION OF POWER HOUSE

PROPOSED EAST CREEK DEVELOPMENT
AT
INGHAM MILLS NEW YORK
G. W. Bick A. J. Denny G. D. Lettermann
THESIS
17



SIDE ELEVATION OF POWER HOUSE

PROPOSED EAST CREEK DEVELOPMENT
AT
INGHAM MILLS NEW YORK
G W Buck, H. J. Devney, G. D. Lettermann
THESIS
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